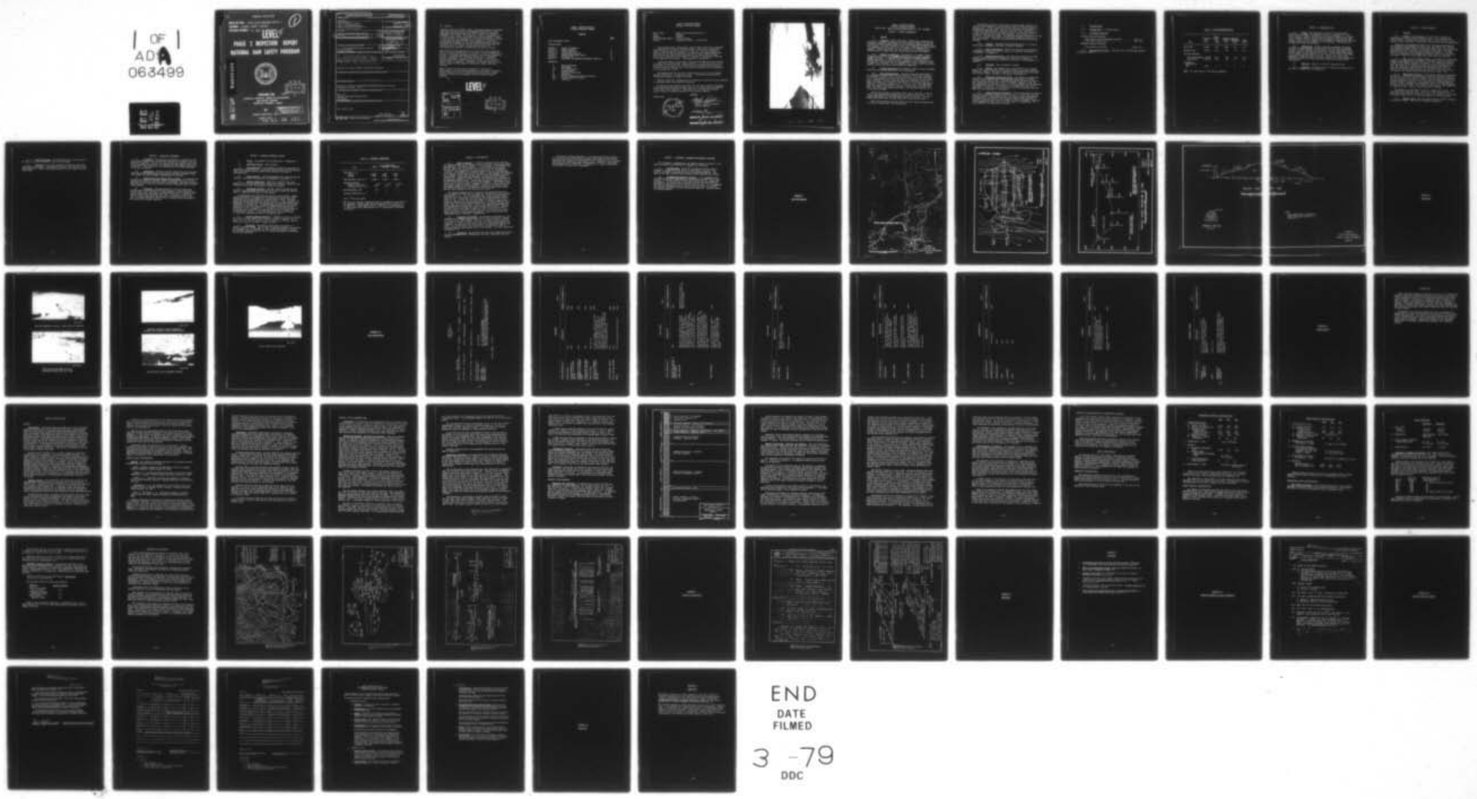
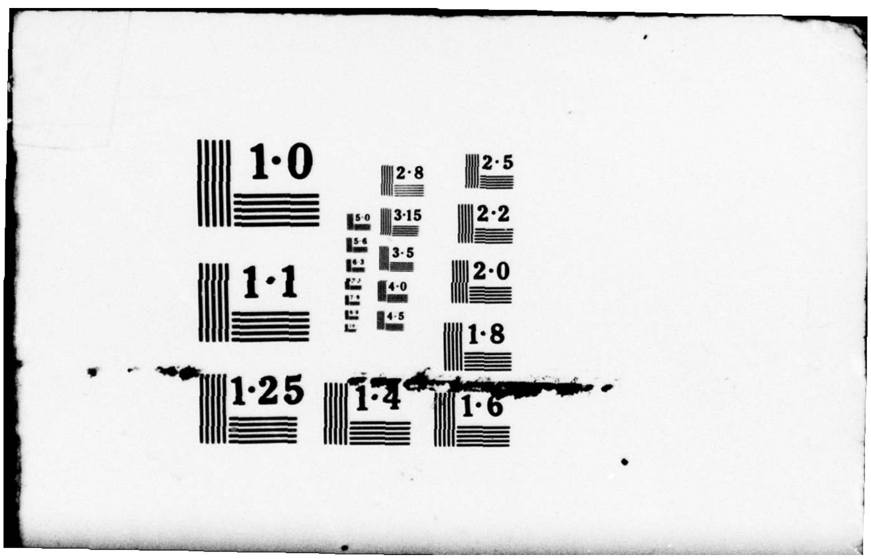
AD-A063 499 GILBERT ASSOCIATES INC READING PA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. UPPER CLINCH WATERSHED DAM NUMBER --ETC(U)
AUG 78 T E ROBERTS DACW65-78-D-0014

UNCLASSIFIED NL

| OF |
AD A
063499



END
DATE
FILMED
3 -79
DDC


1·0
2·8
2·5
3·15
2·2
1·1
3·5
2·0
4·0
4·5
1·8
1·25
1·4
1·6

TENNESSEE RIVER BASIN

Name Of Dam: UPPER CLINCH WATERSHED DAM NO. 8

Location: TAZEWELL COUNTY, VIRGINIA

Inventory Number: VA 18501

LEVEL II

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED FOR

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

GILBERT ASSOCIATES, INC.

AUGUST, 1978

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
VA 18501

2. GOVT ACCESSION NO.

3. RECIPIENT'S CATALOG NUMBER

4. TITLE (and Subtitle)
Phase I Inspection Report
National Dam Safety Program
Upper Clinch Watershed Dam No. 8
Tazewell County, Virginia

5. TYPE OF REPORT & PERIOD COVERED
(9) Final rept.

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
Gilbert Associates ~~Thomas Roberts~~
(10) Thomas E. Roberts

8. CONTRACT OR GRANT NUMBER(s)
(15) DACW 65-78-D-0014

9. PERFORMING ORGANIZATION NAME AND ADDRESS

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
(12) 75p.

11. CONTROLLING OFFICE NAME AND ADDRESS
U. S. Army Engineering District, Norfolk
803 Front Street
Norfolk, VA 23510

12. REPORT DATE
(11) August 1978

13. NUMBER OF PAGES
67

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)

(6) National Dam Safety Program. Upper Clinch Watershed Dam Number 8 (VA18501), Tennessee River Basin, Tazewell County, Virginia. Phase I Inspection Report.

15. SECURITY CLASS. (of this report)
Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service, Springfield, Virginia 22151

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dams - VA
National Dam Safety Program Phase I
Dam Safety
Dam Inspection

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

(See reverse side)

154850

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

LEVEL II

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

CONTENTS

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Upper Clinch Watershed Dam No. 8
State:	Virginia
County:	Tazewell
USGS Quadrangle Sheet:	Tiptop, Virginia
Stream:	Lincolnshire Branch - Clinch River

This recently built, multipurpose, 58-foot high earthen dam is designed to accommodate 271 acre-feet of flood storage. There was a considerable amount of engineering data available for this dam. This data appears to indicate that the dam was designed in accordance with standard engineering practices and was constructed essentially in accordance with the plans. There were no observed critical signs of distress or instability of the dam or its appurtenant structures. (See Appendix IX, Conditions.)

Calculated values of the probable maximum flood (PMF) show that the dam is capable of passing the PMF. Design stability calculations show that the side-slopes of the dam will be stable during both steady-seepage and rapid drawdown conditions coupled with a horizontal earthquake force corresponding to an acceleration of 0.1g.

The abutment and the reservoir valley slopes, as well as the emergency spillway side-slopes, are apparently stable. Foundation conditions were investigated during the design of the dam.

Based on this prior investigation the foundation conditions are not expected to adversely affect the stability of the dam.

An operation and inspection procedure for the gates should be developed within three months because the gates were not in readily operable condition at the time of the inspection. These gates will be required to operate should a need for emergency drawdown of the reservoir develop.

Prepared By:

COMMONWEALTH OF VIRGINIA
JAMES A. HAGEN
No
8349
CERTIFIED PROFESSIONAL ENGINEER

James A. Hagen

APPROVED:

Douglas L. Haller

Douglas L. Haller
Colonel, Corps of Engineers
District Engineer

23 Aug 78
Date

Submitted By: James A. Walsh

Recomended By: James M. Goodwin

June 1978

OVERVIEW PHOTO - UPPER CLINCH WATERSHED DAM NO. 8

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
NAME OF DAM: Upper Clinch Watershed Dam No. 8 ID #: VA 18501

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the U.S. Army Corps of Engineers to initiate a national program of safety inspections of non-Federal dams throughout the United States. The Norfolk Office of the Corps of Engineers has been assigned the responsibility of the inspection of the dams in the Commonwealth of Virginia. Gilbert Associates, Inc. has entered into a contract with the Norfolk Office to inspect this dam, Gilbert Work Order No. 06-7250-003.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (Reference 1 of Appendix VI) and contract requirements between Gilbert Associates, Inc. and the Corps of Engineers. The objectives are to expeditiously identify whether this dam apparently poses an immediate threat to human life or property, and to recommend future studies and/or any obvious remedial actions that may be indicated by the inspection.

1.2 Project Description

The following is based on project documents which were spot checked during the field investigation:

1.2.1 Dam and Appurtenances: The Upper Clinch Watershed Dam No. 8 is a homogeneous earthfill dam, 58 feet high and 670 feet long (including the emergency spillway), with its 18-foot wide top at elevation 2,460 feet m.s.l. The upstream face of the dam slopes 2.5 horizontal:1 vertical above and 3 horizontal:1 vertical below the normal-pool elevation and has two berms; the lower one at elevation 2,427 feet m.s.l. is 10 feet wide and the upper one at elevation 2,436 feet m.s.l. is 12 feet wide. The downstream face also slopes 2.5 horizontal:1 vertical, but has only one 10-foot wide berm. This berm slopes between elevations 2,431 feet and 2,428 feet m.s.l. to facilitate drainage.

The dam does not have a horizontal filter, but if does have a vertical filter drain intercepting drainage from the foundation-soil strata. It is located approximately 40 feet upstream of the toe of the dam. A 10-inch diameter perforated pipe is placed within the top part of the drain.

The drawings indicate that the positive cut-off, having a minimum width of 12 feet, is taken down to the firm bedrock.

The principal spillway, located near the left abutment, consists of a concrete-cradle supported, 30-inch internal diameter, reinforced-concrete pressure pipe. It has a reinforced-concrete drop-inlet and discharges on a plunge-pool energy dissipator. The top of the drop-inlet structure is 12 feet lower than the crest elevation of the dam. A boat and ladder must be used to reach the top of the inlet structure. The structure is not connected to the dam by a bridge. The emergency spillway (elevation 2,447 feet m.s.l.) is a 55-foot wide trench cut through the weathered rock strata at the right abutment. A 180-foot long dike forms the left-side slope of the spillway.

1.2.2 Location: The Upper Clinch Watershed Dam No. 8 is located about 2 miles northeast of the town of Tazewell, Virginia.

1.2.3 Size Classification: The size of this dam is classified as "intermediate" based on its 60-foot height in accordance with Section 2.1.1 of Reference 1 of Appendix VI.

1.2.4 Hazard Classification: This dam can be placed in the "high hazard" category because of its proximity to the town of Tazewell, in accordance with the guidelines contained in Section 2.1.2 of Reference 1 of Appendix VI.

1.2.5 Ownership: Town of Tazewell, Virginia.

1.2.6 Purpose: The intended uses of the reservoir are to supply water to the town of Tazewell and to provide fishing, recreation, and flood prevention. However, the town of Tazewell has not needed to draw this water for its water supply, according to the owner's representative.

1.2.7 Design and Construction History: The dam and its appurtenant structures were designed by Harza Engineering Company (HEC) of Chicago, Illinois, during 1970 and 1971. HEC submitted the Design Report to the U.S. Soil Conservation Service (SCS) office in Richmond, Virginia, in May 1971. The construction was carried out by the C. S. Horton Company and was supervised and inspected by the SCS. The construction was completed in July 1972. Reportedly, no modification or alteration to any of the structures has been made since the completion of construction.

1.2.8 Normal Operational Procedures: During floods when the reservoir water level rises above elevation 2,436.0 feet m.s.l., water flows into the principal spillway through open spaces in the inlet and hence the operation is uncontrolled. At or below this elevation, however, gate operation is required to withdraw water for water-supply or reservoir drawdown. Operation of the emergency spillway, which is an open trench, is uncontrolled. No other operations are required for this dam.

1.3 Pertinent Data

1.3.1 Drainage Area: 1.95 square miles.

1.3.2 Discharge at Dam Site:

Principal Spillway Discharge:

Pool level at crest of emergency spillway . . . . . . 130 c.f.s.
Pool level at top of dam . . . . 140 c.f.s.

Emergency Spillway Discharge:

Pool level at top of dam . . . . 8,560 c.f.s.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are given in Table 1.1

Table 1.1 DAM AND RESERVOIR DATA

Item	Elevation feet m.s.l.	Water Surface Area acres	Reservoir Capacity Acre feet	Reservoir Capacity Watershed inches	Length miles
Top of Dam	2,459.7	44.5	1,050	10.1	0.9
Spillway Crest	2,446.9	29.8	576	5.5	0.8
Principal Spillway					
Low Stage Opening	2,436.00	20.6	305	2.9	0.6
Riser Crest	2,445.65	27.6	535	5.1	0.8
Streambed at Centerline of Dam	2,402	0	0	0	0

NOTE: For other details on the dam see Appendix I.

SECTION 2 - ENGINEERING DATA

2.1 Design: The design data is given in the Design Report (Reference 6 of Appendix VI) mentioned earlier in paragraph 1.2.7. The report contains a general description of the project, summary of the report, layout drawings, hydraulic data and design, subsurface and soil-test data (part of which is given in Appendix IV), embankment and appurtenance-structure designs, quantities, bid schedule, and cost estimates. SCS criteria and procedures were used in the design.

2.2 Construction: As-built construction drawings and records including the specifications are available at the SCS office in Richmond, Virginia. As-built drawings received by GAI show that some changes in the original plans were made during construction. Notable among these changes was the change in the planned use of rock-fill in the 8.5-foot thick Zone-2 layer on the downstream slope of the dam. Instead, the dam was built homogeneous, using the clayey silty soil in Zone-2 also. Other changes were basically related to the actual subsurface conditions revealed during construction.

2.3 Operation: There is no record of operational data.

2.4 Evaluation: Available records on design and construction of this dam are adequate for this evaluation.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

3.1.1 General: The topography of the dam site is gentle with rounded-top hills surrounding the reservoir area. A flat and wide area which includes a recreation area stretches from the toe of the dam to State Highway 61 about one-third of a mile away. Thus, the dam is easily approachable.

Operational and maintenance inspections of this dam were performed by SCS personnel during December 1974, April 1976, and October 1976. The SCS also performed a safety inspection of the dam in April 1978. These inspections (see Appendix VI) did not reveal any significant deficiencies or unusual behavior of the dam. During the safety inspection, however, it was found that the drawdown-gate hoist assembly was displaced and that no attempt had been made to operate the water-supply gates since they were installed.

3.1.2 Dam: No sign of unusual behavior of the dam such as downstream seepage, uneven settlement, displacement, sloughing of slopes, cracking or seepage at dam-abutment junctions was observed. The slopes were adequately covered with protective vegetation, except for two narrow pathways on the dam. The riprap on the upstream slope was generally in good condition. A hand-level measurement from the water surface indicated that the crest elevation of the dam, where measured, was 2,461 (±0.5) feet; that is, more than one-half foot higher than the settled elevation (2,459.7 feet).

3.1.3 Appurtenant Structures: The principal spillway was apparently working well. Some flow was seen passing through this spillway indicating that the reservoir water level was slightly above the normal-pool elevation. The water emerging at the outlet was clear. The water-supply gates were not operated during this inspection. The drawdown-gate stem-casing appeared slightly crooked, although the stem appeared vertical. The operating wheels were not on the valve stems. The condition of the riprap at the plunge-pool was apparently good. The pool area was enclosed by fencing.

The emergency spillway showed no signs of unusual deterioration. Both side slopes were apparently stable. The crest of the spillway and its downstream channel are used as the access road to the reservoir. The crushed rock spread on the crest of the spillway appeared to be well maintained.

3.1.4 Reservoir Area: The side slopes along the reservoir periphery are gentle and stable. The slopes are adequately vegetated.

3.1.5 Downstream Channel: The downstream area in the vicinity of the dam is flat and it provides a wide and clear channel.

3.2 Evaluation: The visual inspection revealed no significant deficiencies in the dam, its appurtenant structures, the reservoir or the downstream areas. However, the operating wheels were not found on the gate riser stems.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: No operational procedures are required for this dam during flooding because the spillway operations are automatic above the normal-pool elevation. In case of a need to lower the reservoir water level below the normal-pool elevation, the drawdown and water supply gates have to be operated. This operation is the responsibility of the Water Department of the town of Tazewell.

4.2 Maintenance: Except for periodic operational and maintenance inpections by SCS and subsequent compliance by the Town of Tazewell, there is no formal and regular program of maintenance for this dam.

4.3 Description of Any Warning System in Effect: No sophisticated warning system exists at this dam. There is a building at the toe of the dam which is staffed by the Town Recreation Department. They have reportedly been instructed to notify the town and its police department should a hazard become apparent.

4.4 Evaluation: Although operation of the water supply and drawdown gates is not required for flood discharge, it may be required to draw the water level down should a problem develop with the dam which would require repairs below the normal water level. Therefore, it should be verified within three months that the gates can be operated and, at that time, a procedure developed, placed on file, and made readily available for gate operation and annual inspection.

SECTION 5 - HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: See Appendix VI-B and Reference 6 of Appendix VI.

5.2 Hydrologic Records: None available.

5.3 Flood Experience: The emergency spillway has reportedly not experienced any flows over it. During the flood of April 1977, the reservoir water level reached elevation 2442.5 feet according to the Owner's representatives.

5.4 Flood Potential: Various hydrographs were routed through the reservoir, the results are presented in paragraph 5.6 and Table 5.1.

5.5 Reservoir Regulation: There are no controls other than manual valves to lower the pool below normal-pool elevation. Flows over the principal spillway and the emergency spillway are automatic.

5.6 Overtopping Potential: The PMF, one-half the PMF, and the 100-year flood hydrographs were developed and routed through the reservoir. Table 5.1 summarizes the results of the analyses.

The hydrographs were developed and routed by using the HEC-1 computer program (Reference 2 of Appendix VI) and appropriate precipitation, unit hydrograph, and storage-volume versus outflow data as input. The three inflow and outflow hydrographs are listed in Table 5.1. The triangular unit hydrograph was developed from the drainage area and estimated time to peak (Reference 3 of Appendix VI). Probable maximum precipitation and 100-year precipitation data were obtained from U.S. Weather Bureau publications (References 4 and 5 of Appendix VI). Information from design drawings and the design report (Reference 6 of Appendix VI) was used to compute the storage-outflow relation. Losses were estimated at an initial loss of 1.0 inch and a constant loss rate of 0.48 inch per hour.

5.7 Reservoir Emptying Potential: Assuming all valves are operable, the 30-inch concrete conduit will be able to provide a withdrawal rate of about 126 c.f.s. with the reservoir level at elevation 2,445 feet, and will dewater the reservoir in about 2.6 days (63 hours).

5.8 Evaluation: The results indicate that the reservoir is capable of passing the PMF. The spillway is considered adequate in accordance with paragraph 3.5.1 of Reference 1. This conclusion pertains to present day watershed conditions and the effect of future developments on hydrology has not been considered.

Table 5.1 - RESERVOIR PERFORMANCE

Item	Flood Hydrograph PMF	1/2 PMF	100-Year
Peak Flow, c.f.s.			
Inflow	10,800	5,380	2,390
Outflow	8,950	4,260	369
Peak Elevation, feet m.s.l.	2,459.8	2,455.1	2,448.3
Emergency Spillway			
Depth of flow, feet	8.3[a]	5.2[a]	0.9[a]
Average velocity, f.p.s.	14.8	12.1	5.1
Dam Overtopping, feet	0.1*	--	--
Tailwater Elevation, m.s.l.	--	--	--

Note: [a]Critical depth.

**This does not consider a reduction factor in the PMF due to the size of* the watershed. The peak PMF inflow furnished by SCS is 10,134 c.f.s. as compared to 10,800 computed by GAI. Therefore, an overtopping of 0.1 feet being within computational error is disregarded in the final conclusions.

SECTION 6 - DAM STABILITY

6.1 Stability Analysis: A stability analysis for the upstream and downstream slopes of the dam was carried out by Harza Engineering Company (see Appendix V) and appears adequate. In the analysis, it was assumed that both the slopes were 2.5 horizontal to 1 vertical whereas the drawings indicate that on some portions of the dam the slope is 3.0 horizontal to 1 vertical. The assumption of uniform strength of the fill and foundation soils (800 psf cohesion and 24° friction angle) is valid because the foundation soil thickness is very small (about 2 feet) on the left abutment and at the valley bottom. Toward the right abutment, the foundation soil thickness increases to a maximum of 15 feet, but the rock elevation also increases and the embankment height reduces. The borings show that the foundation soil is "stiff" to "very stiff" silty clay with rock fragments. The absence of any significantly large settlement, sloughing or failure of slopes confirms that the assumed strength is not unreasonable. Laboratory test results on the undisturbed foundation-soil and compacted borrow-area samples also showed that the shear strength of both the soils were similar.

The computed minimum factor of safety for the upstream slope with a rapid drawdown of 34 feet from the normal-pool elevation and 0.1g horizontal acceleration is 1.77. For the downstream slope, with the steady-seepage condition and 0.1g horizontal acceleration, the minimum computed factor of safety is 1.81. If the clay fill was assumed for the Zone 2 also in the stability computation for the downstream slope, a slightly higher factor of safety than 1.81 would have been obtained due to the clayfill weight being smaller than that of the rockfill. Harza's own computer program based on the Morgenstern-Price method was used to analyze the stability. Although, the input details other than the geometry of the sections and the soil properties were not available to GAI, the values of the factors of safety appear reasonable for the conditions analyzed. These values satisfy the requirements given in Reference 1 (Appendix VI).

6.2 Foundation and Abutment: As mentioned in paragraph 6.1, the foundation-soil thickness varies from 2 feet on the left side to a maximum of 15 feet at the right abutment. The soil is largely residual in nature, consisting of "stiff" to "very stiff" silty clay containing decomposed rock fragments. The underlying rock strata consist of interbedded limestone and shale. The top few feet of the rock surface is weathered but tight as indicated from the water-pressure tests in the borings. (For more information, see Appendix IV.)

6.3 Evaluation: The available data and visual inspection indicate that the embankment and its foundation do not pose a significant stability problem.

This dam is located within Zone 2 on the Algermissen Seismic Risk Map of the United States (1969 edition) and the visual inspection and studies described herein indicate the dam apparently has satisfactory static stability conditions and conventional safety margins exist. Therefore, in accordance with paragraph 3.6.4 of Reference 1 of Appendix VI, it may be assumed the dam presents no hazard due to earthquakes.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS/REMEDIAL MEASURES

The assessment, recommendations and remedial measures contained in this Report are based on the provisions of Appendix IX, Conditions.

7.1 Dam Assessment: There were no observed critical signs of distress or instability of the dam or its appurtenant structures. The spillway is capable of passing the PMF and is considered adequate. The gates at the inlet structure have not been operated regularly nor recently.

7.2 Recommendations/Remedial Measures: It is recommended that the drawdown and water-supply gates of the principal spillway inlet be made operable. It should be verified within three months that the intake gates are operable. At the same time, a procedure for gate operation should be developed in consultation with the SCS and placed on file. It is recommended that the gate operating wheels be placed on the gate stems and connected with chains and padlock to prevent unauthorized operation. The gates should be inspected annually.

APPENDIX I

MAPS AND DRAWINGS

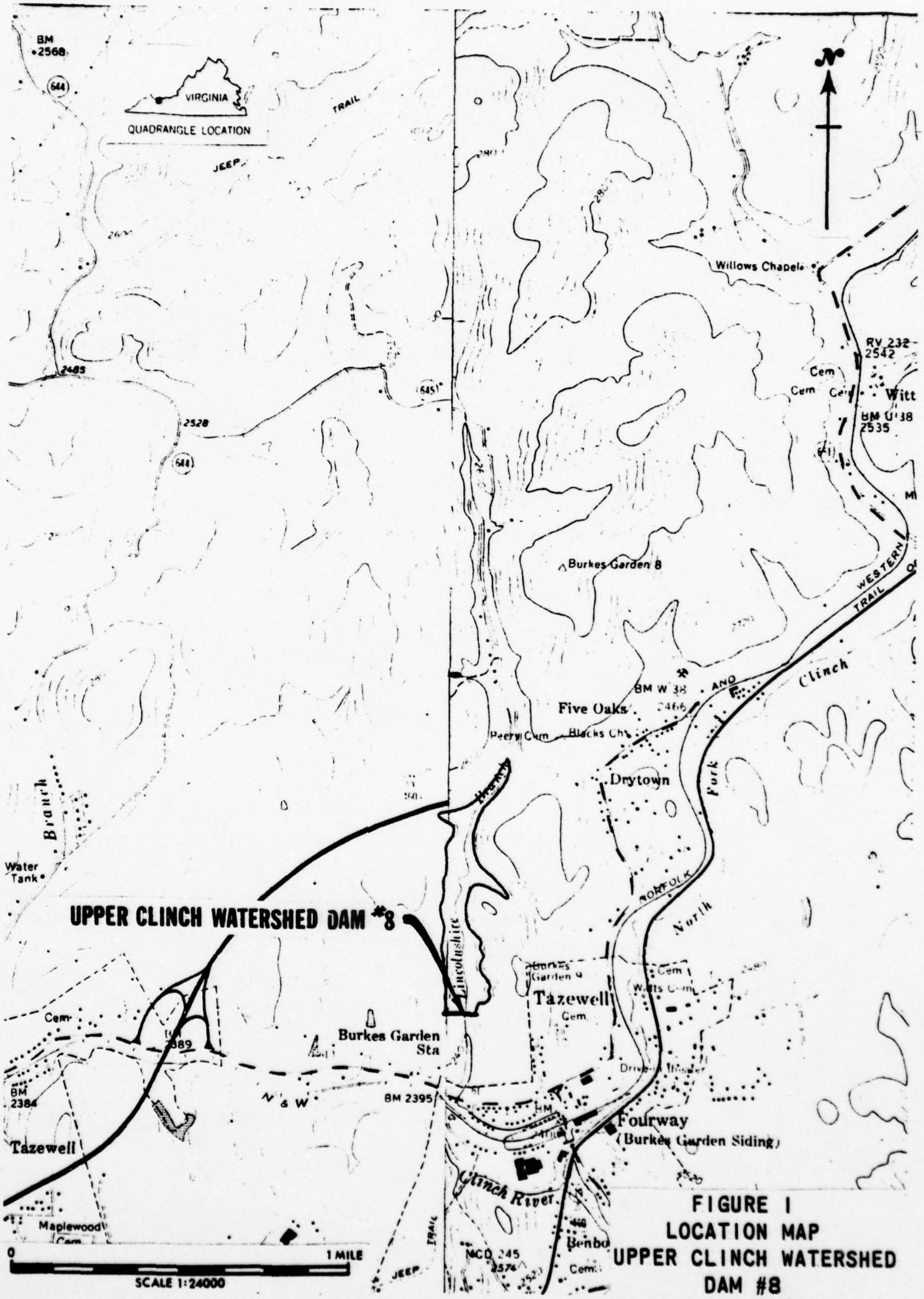


FIGURE I
LOCATION MAP
UPPER CLINCH WATERSHED
DAM #8

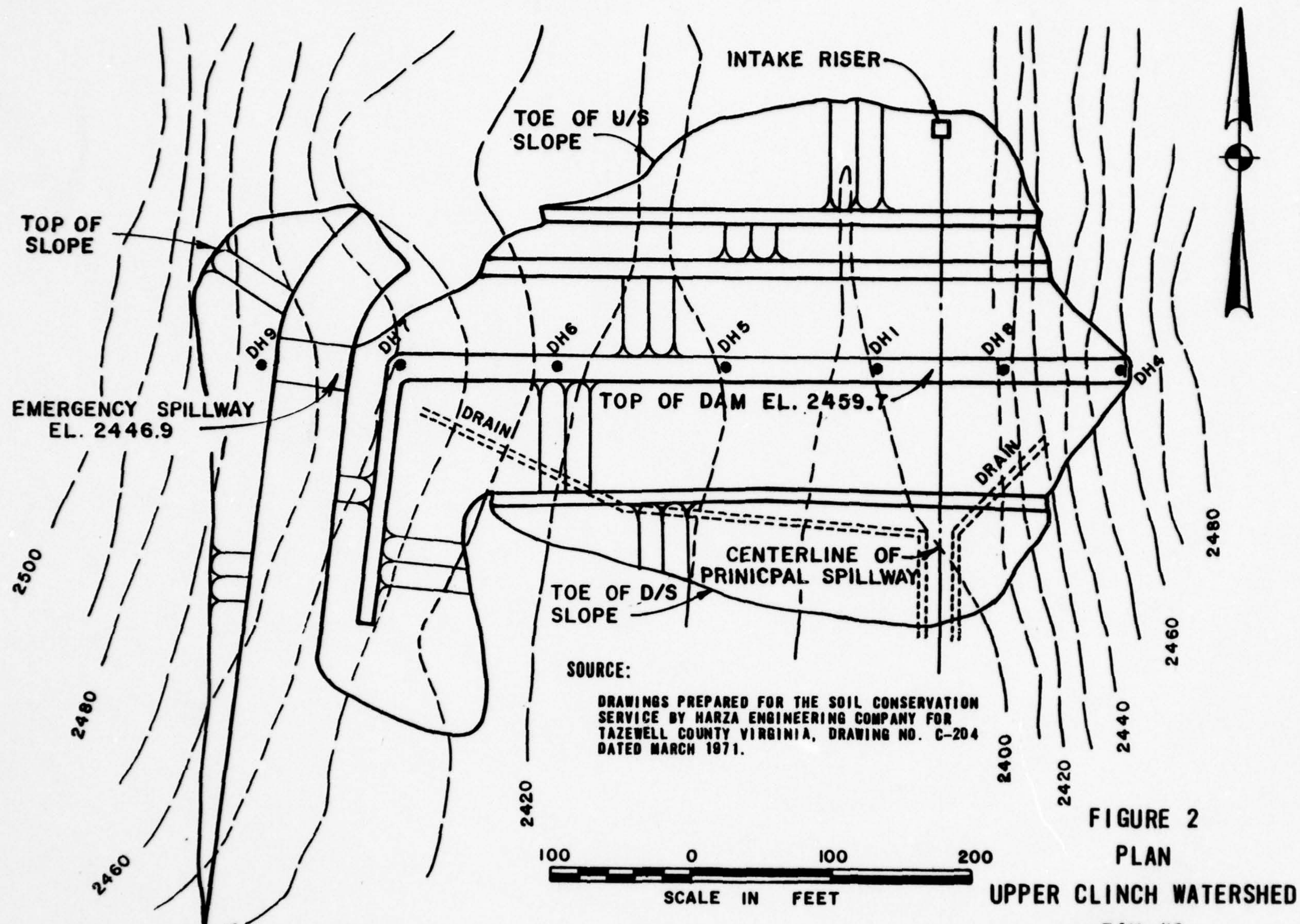
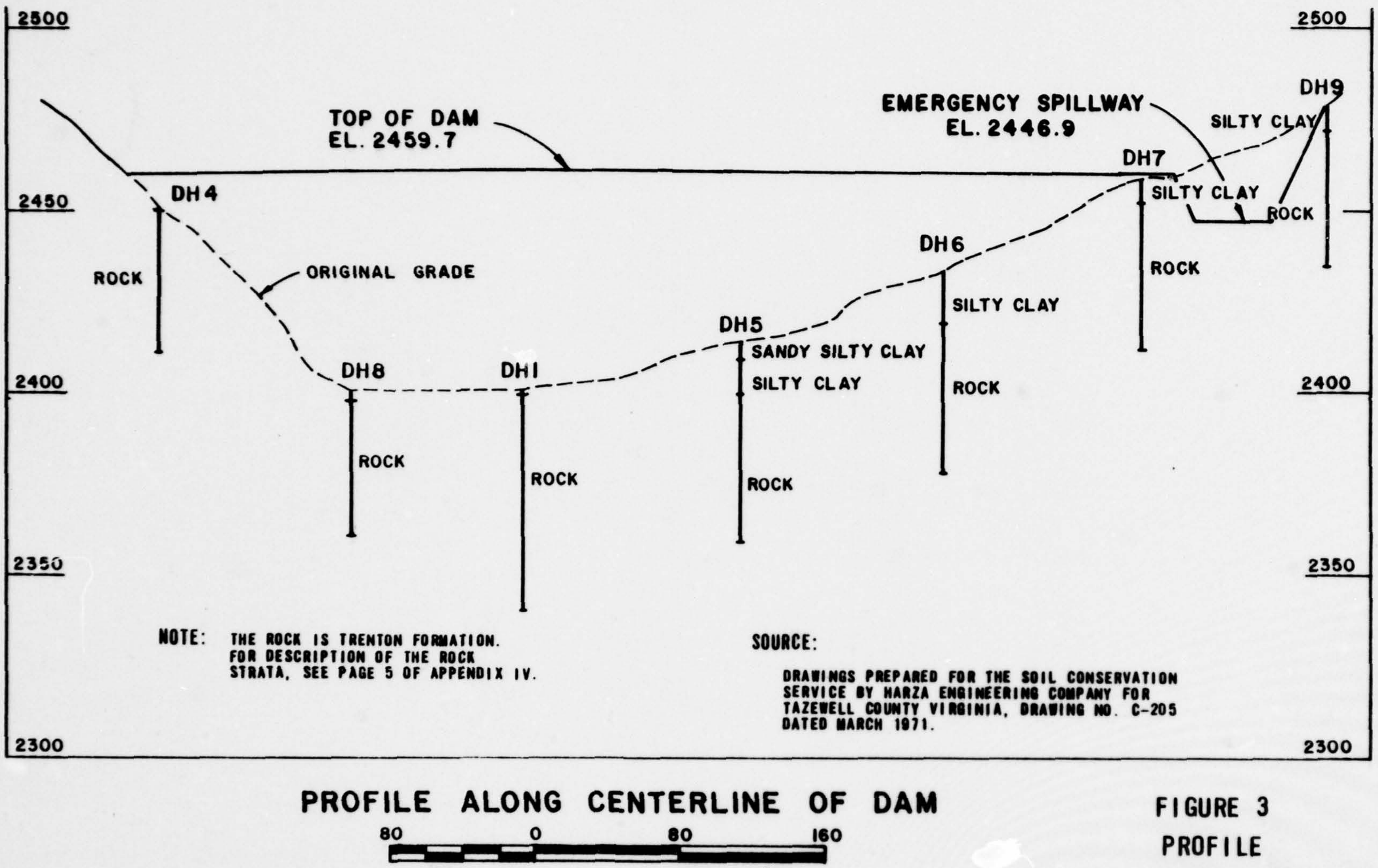
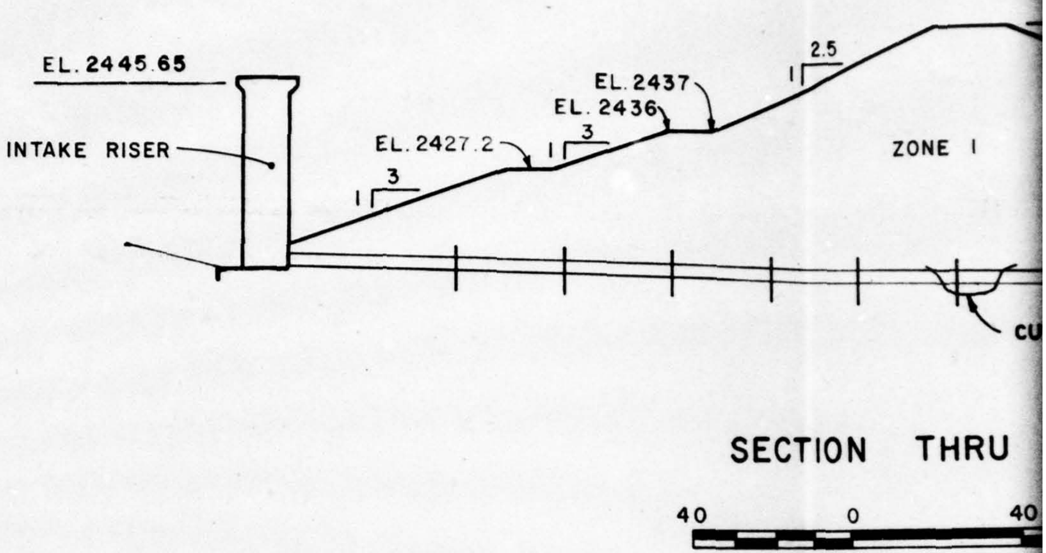


FIGURE 2
PLAN
UPPER CLINCH WATERSHED
DAM #8

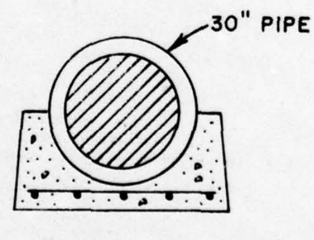


PROFILE ALONG CENTERLINE OF DAM

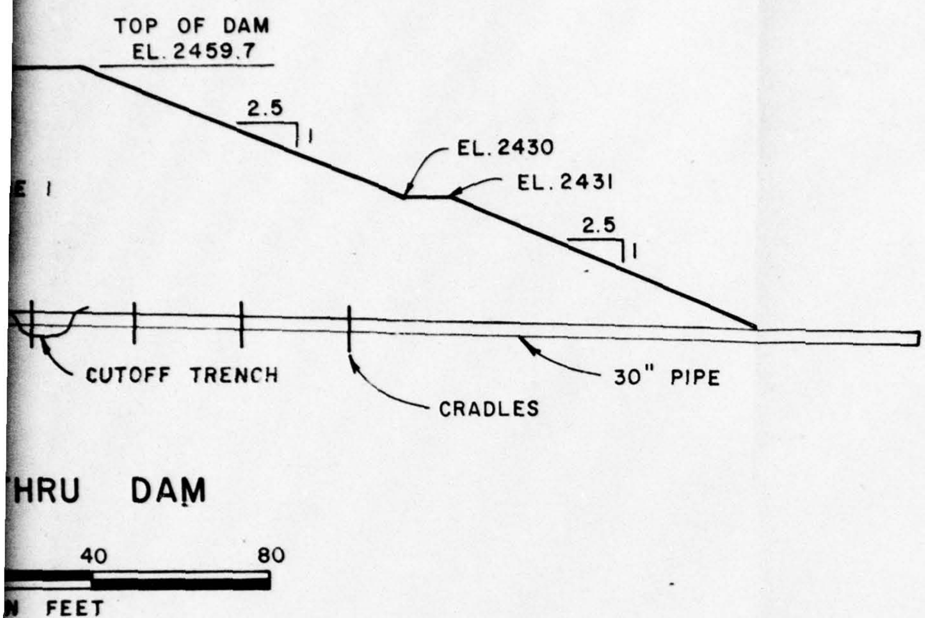
FIGURE 3
PROFILE
UPPER CLINCH WATERSHED
DAM #8



SECTION THRU



CRADLE DETAIL
NO SCALE



SOURCE:

DRAWINGS PREPARED FOR THE SOIL CONSERVATION SERVICE BY HARZA ENGINEERING COMPANY FOR TAZEWELL COUNTY VIRGINIA, DRAWING NO. C-207 DATED MARCH 1971.

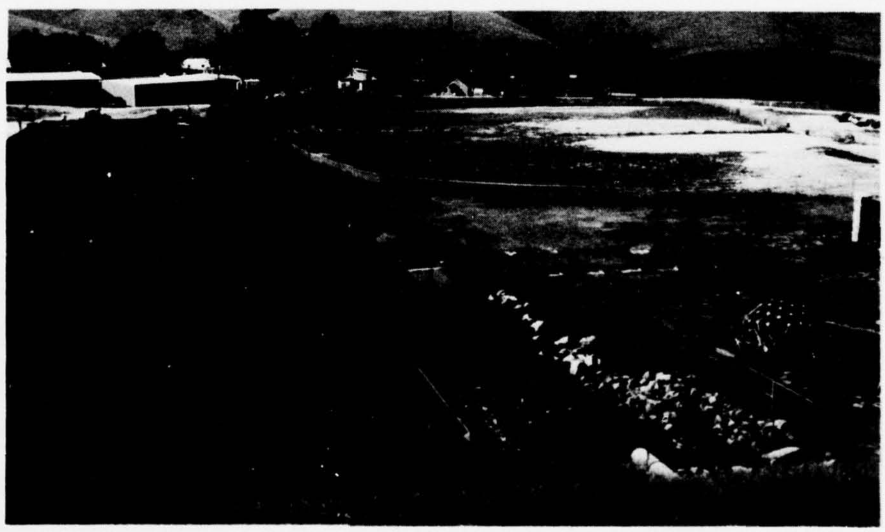
FIGURE 4
SECTION AND DETAIL
UPPER CLINCH WATERSHED
DAM #8

APPENDIX II

PHOTOGRAPHS

June 1978

VIEW FROM EMERGENCY SPILLWAY - NOTE RIPRAP AT WATERLINE



June 1978

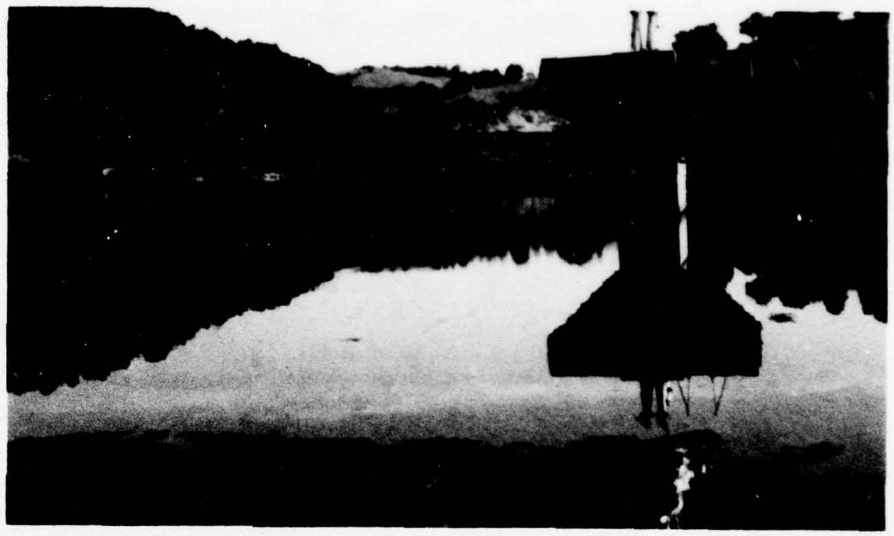
VIEW OF RIVER DISCHARGE POINT AND
DOWNSTREAM CHANNEL FROM TOP OF DAM

June 1978

EMERGENCY SPILLWAY LOOKING DOWNSTREAM;
BUILDING IN FOREGROUND IS FOR RECREATION ONLY

June 1978

VIEW OF RIGHT SIDE OF EMERGENCY SPILLWAY



June 1978

VIEW OF INTAKE RISER FROM DAM

APPENDIX III

FIELD OBSERVATIONS

Check List
Visual Inspection
Phase 1

Name Dam: Upper Clinch Watershed Dam No. 8 County: Tazewell State: Virginia Coordinators: Norfolk District Corps of Engineers

Date Inspection: 15 June 1978 Weather: Sunny and Clear Temperature: 70°F

Pool Elevation at Time of Inspection: 2,436.5 feet m.s.l. Tailwater at Time of Inspection: Not Applicable

Gilbert Associates, Inc.
Inspection Personnel:

James A. Hagen
Nazir A. Qureshi
Yogesh S. Shah

Also Present:

Mr. C. H. Leist, Town Manager
Mr. Bain, Town Recreation Dept.
Duncan McGregor, U.S. Soil Conservation Service (SCS)
Buck Arnold - Virginia State Water Control Board

James A. Hagen - Recorder

EMBANKMENT

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None	None
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None	None
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None	None
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Good	None
RIPRAP FAILURES	None Observed	None
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No sign of seepage or erosion was found at the left abutment-dam junction. Junction of the dam at right with the cross dike was also in satisfactory condition. Junction of the principal spillway and dam at the outlet end did not show significant erosion. The outlet pipe projects outward sufficiently.	None
ANY NOTICEABLE SEEPAGE	None	None
STAFF GAGE AND RECORDER	None	None
DRAINS	The drains apparently function well.	None

OUTLET WORKS

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	None	None
INTAKE STRUCTURE	The outlet work consists of a 30-inch diameter reinforced concrete pressure pipe running through the dam and a drop-inlet entrance. It also works as the principal spillway because it is intended to discharge flood water when its operation is uncontrolled. For water levels at or below the normal pool elevation, gate operation is required. The riser structure showed no signs of significant deterioration condition. The water supply gates were not operated. The drawdown gate stem-casing was slightly crooked, but the stem appeared vertical. There were no operating wheels on the valve stems.	The operation of the gates must be verified and the gates fixed if necessary.
OUTLET STRUCTURE	The outlet end of the pipe showed no significant deterioration. No significant erosion around it was seen. The plunge-pool riprap showed no signs of failure. The area was fenced.	None

OUTLET WORKS

Sheet 2

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
OUTLET CHANNEL	The outlet channel was clear. There were no signs of unusual erosion.	None
EMERGENCY GATE	Not Applicable	

UNGATED SPILLWAY

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	There was some indication of rock or rough concrete under the gravel surface of the spillway crest. This area appeared to have good alignment.	None
APPROACH CHANNEL	The emergency spillway is a 55-foot wide open trench at the right abutment of the dam. The riprap and the crushed rock bed underlain by rock appeared to be well maintained.	None
DISCHARGE CHANNEL	The cross dike providing the left side slope was stable and the riprap was in good alignment. The right side slope cut through the weathered rock was also apparently stable. The bed was used as the road and was satisfactorily maintained.	None
BRIDGE AND PIERS	Not Applicable	

INSTRUMENTATION

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	Not Observed	None
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	None	

RESERVOIR

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Slopes were in stable condition. Some small exposed soil areas were seen, which were apparently used as the borrow areas during the dam construction. These areas appeared stable.	None
SEDIMENTATION	Excessive sedimentation was not apparent.	None

DOWNSTREAM CHANNEL

Sheet 1

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	There were no obstructions which would cause a significant tailwater to develop against the dam. (See photographs in Appendix II)	
SLOPES	Gentle to flat.	
APPROXIMATE NO. OF HOMES AND POPULATION	There appear to be fewer than 10 homes in the channel before it enters the Clinch River about 2 miles upstream of the town of Tazewell, Virginia, which has a population of about 4,500 people.	

APPENDIX IV

GEOLOGY REPORTS

INTRODUCTION

Harza Engineering Company was retained by the U.S. Soil Conservation Service to perform the foundation-materials investigation and design of a multi-purpose dam, dam #8, on the Lincolnshire Branch of the Clinch River in Virginia. The project is located two miles northeast of the town of Tazewell, the county seat of Tazewell County, in southwest Virginia. The project consists of a 500 foot long earth embankment 60 feet high, a drainage area of about 2 square miles, and a reservoir with 150 acre feet of municipal water supply storage plus about 271 acre feet of flood storage.

The purpose of the geologic investigations was to determine the feasibility of the site for construction of the dam and for the storage of the impounded water. The scope of the investigations consisted of a review of existing geologic data; reconnaissance and geologic mapping of the damsite, reservoir, watershed and adjacent areas; subsurface exploration of overburden, bedrock and groundwater; and laboratory testing.

GEOLOGIC INVESTIGATIONS

GENERAL

Physiography: The project area is located in the Valley and Ridge province, which is part of the Appalachian Highlands, a physiographic feature that extends in a generally northeast-southwestward direction along the eastern North American continent from Newfoundland to Alabama. From east to west, these Highlands consist of the Piedmont Province, the Blue Ridge Province, the Valley and Ridge Province and the Appalachian Plateau Province. The Valley and Ridge Province consists of a belt of parallel ridges and valleys trending in an east-northeasterly direction in the project area and in a more northeasterly direction in other parts of the province. In the Upper Clinch Valley, the topography is moderately rugged. The most prominent ridges are Buckhorn Mountain (4196) feet and Rich Mountain (4245 feet). Elevations in adjacent valleys are on the order of 2400 to 2700 feet.

Several cycles of erosion to a low level (peneplain) followed by renewed uplift are recognized in the Valley and Ridge province. The oldest peneplain is relatively obvious in the general coincidence of the tops of the more prominent ridges. Other and less prominent topographic features mark more recent peneplains. Drainage in the province is preponderantly parallel to the ridges and valleys, however, several major streams course through the province transversely and could reflect a drainage pattern developed on a peneplain and subsequently superimposed on rising terrain. The major streams that cross the province transversely include the Potomac, James, New, and Tennessee Rivers. A few tributaries to the Clinch River in the western part of the watershed trend in a north-south direction and also are suggestive of a superimposed drainage pattern. One of these tributaries is the Lincolnshire Branch upon which site 8 is located.

Geologic Setting: The sedimentary rocks underlying the Valley and Ridge province were deposited in an elongate shallow sea during the Paleozoic Era. Upwards of perhaps 40,000 feet of sediments were deposited in the slowly sinking geosyncline. At the end of the Paleozoic Era, the geosyncline was uplifted, and the sediments folded into northeast-southwestward trending anticlines and synclines which the processes of erosion subsequently modified into the ridges and valleys described earlier.

The Upper Clinch Valley is underlain by sedimentary rocks that range in age from Cambrian through Mississippian, and consist of limestone, dolomite, shale, siltstone and sandstone. The highest ridges in the valley occur in the center of synclines. The tops of many of these peaks are formed of Clinch (Tuscarora) sandstone, a hard and resistant quartzite. Lower elevations in the valley are underlain by shales, limestones and other less resistant rocks.

Faulting is a prominent feature of the Upper Clinch Valley watershed; two of the most prominent faults are the St. Clair and the Narrows thrust faults. These two faults form the northern and southern boundaries of the East River Mountain Synclinorium. Site #8 is located on the northern flank of the synclinorium.

Site #8 is on the Lincolnshire Branch of the Clinch River, about 1500 feet upstream from the confluence of the branch and the main stream (Exhibit 1). The Lincolnshire Branch flows south, transverse to the east-northeast geologic trend. As described earlier, the stream pattern appears to have been established on an ancient peneplain surface. This surface is represented by the generally flat and coinciding elevation of the two ridges that border most of the reservoir.

Topography within the project area is low and gentle. From an elevation of 2400 feet in most of the valley bottom, the topography rises to a maximum elevation of 2720 feet adjacent to the reservoir area. The maximum elevation bordering the watershed is about 3040 feet. Slopes in the reservoir area are gentle and reach a maximum of about 20° in limited areas at mid-reservoir and in upper parts of the reservoir.

SCOPE OF FIELD INVESTIGATIONS

Surface: All available literature pertinent to the project area was reviewed, and consisted of the following:

Butts, Charles, Geology of the Appalachian Valley in Virginia, Virginia Geological Survey Bulletin 52, 1940.

Cooper, B. N., Geology and Mineral Resources of the Burkes Garden Quadrangle, Virginia, Virginia Geological Survey Bulletin 60, 1944.

Cooper, B. N., Industrial Limestone and Dolomites in Virginia; Clinch Valley District, Virginia, Virginia Geological Survey Bulletin 66, 1945.

Harsnsberger, T. K., The Geology and Coal Resources of the Coal Bearing Portions of Tazewell County, Virginia, Virginia Geological Survey Bulletin 19, 1919.

Ries, H., and Somers, R. E., the Clays and Shales of Virginia West of the Blue Ridge, Virginia Geological Survey Bulletin 20, 1920.

Surface investigations consisted of reconnaissance geologic mapping of the damsite, reservoir, and surrounding area on an aerial photo print, scale one inch = 300 feet, and on a one inch = 500 feet enlargement of part of the Burkes Garden and Pounding Mill 15 minute topographic quadrangle maps (Exhibit 1). In order to properly define the character of the formations underlying the damsite and the reservoir, it was necessary to map and study

the outcrops both within the limits of the project and in neighboring terrain. Particular importance was given to the mapping of several cavernous limestone formations that underlie the upstream third of the reservoir. The Trenton Formation of shale and limestone that underlies the downstream half of the reservoir plus the damsite does not form prominent outcrops within the project area. Much of the information pertaining to the formation was gained from studying outcrops in road cuts in the surrounding area.

Subsurface: Subsurface investigations consisted of overburden drilling and sampling in the damsite area; core drilling and pressure testing in bedrock in the damsite area and at one locality in the reservoir area; test pitting and sampling in the damsite area and in potential borrow areas; and a test trench in the foundation area. Locations of most of the explorations are shown on Exhibit 2. The locations of a few test pits outside the area shown on Exhibit 2, are shown on Exhibit 1. The log of the test dozer trench is shown on Exhibit 4; the logs of drill holes and test pits are shown on Exhibits 5 and 6 respectively. Permeability and pressure test data are contained on Exhibit 7.

Fourteen borings were made in the dam foundation and emergency spillway areas. Standard penetration tests and split spoon sampling were accomplished in each hole. In addition, two Shelby tube samples of overburden were taken in holes located in the valley floor of the foundation area. Twenty-three test pits were dug with a back hoe in the foundation and spillway areas. The test pits were logged, six undisturbed chunk samples and 17 sack samples were taken for laboratory testing.

The core holes in the dam foundation and emergency spillway area were drilled through the weathered zone and some distance into fresh, tight rock. Constant head permeability and pressure tests were performed on all of the holes except three that were located above the embankment in the emergency spillway area. Two additional core holes were drilled in the reservoir area in order to evaluate the leakage potential along a possibly disturbed geologic contact. One of the holes was drilled at an angle of 45 degrees; the other was drilled vertically. Constant head permeability tests and pressure tests were performed on the two holes.

Forty-two test pits were dug in potential borrow areas within the reservoir, on top of the hill immediately east of the reservoir, and in the corn field on the right valley slope downstream of the embankment. Each pit was logged and a sack sample taken. The pits were dug either to a depth of 11 feet, the maximum capacity of the back hole, or to refusal at a shallower depth.

A 170 foot long test trench was cut along the lower left abutment in order to permit a study of the in-place rock characteristics in the embankment area.

GEOLOGY OF THE EMBANKMENT AREA

Overburden: The overburden in the embankment area is relatively thin, varying from a few inches on the left abutment to an average of 2 or 3 feet in the valley bottom and a maximum of about 15 feet on the right abutment. In the emergency spillway area of the right abutment, the overburden is appreciably thinner, perhaps averaging 5 feet. The overburden is largely residual in nature, consisting of dense silty clay containing much relic rock structure and decomposed rock (primarily shale) fragments.

Bedrock Stratigraphy, Structure and Lithology: Much of the central part of the East River Mountain Synclinorium, on which damsite #8 is located, is composed of shale and limestone of the Martinsburg Formation. The Trenton Division (hereafter referred to as Trenton Formation) forms the basal unit of the Martinsburg and underlies the damsite and most of the downstream half of the reservoir. As it occurs in the project area, the Trenton Formation consists of several hundred feet of thinly bedded argillaceous limestone and calcareous shale (Exhibits 1 and 3). The limestone occurs in beds of less than an inch to a maximum of 2 feet in thickness, while the shale, which composes 15-20 percent of the content of the formation, occurs in much thinner beds and lenses. Because of its argillaceous and thin bedded nature, the formation is very susceptible to surface weathering and does not form prominent natural outcrops. Within the project area a few small outcrops and near-outcrops occur in the bottom of the stream channel, on the left abutment, and in the emergency spillway area at the right abutment. Road cuts, the excavated test trench, and other artificial exposures offer the only opportunity for a detailed study of the formation in-place.

As seen in outcrops, the rock strata in the Trenton Formation are wavy, discontinous, and generally occur in tight chevron folds having high angle axial planes. Shale flowage is evident in the apexes of some of the folds. Faults having small offsets and containing some gauge can be seen in the larger exposures. The overall formational dip of the Trenton within the project area is about 40° to the south-southeast, and the strike is about N 65°E. Exhibit 4 is a geologic profile of a 170-foot long test trench cut nearly perpendicular to the bedding strike along the lower left abutment. The profile illustrates the close folding in the formation.

The clay mineral content of the Trenton limestone varies and causes the color of the rock to range from light to dark gray. The limestone is generally medium grained but varies from fine to coarse. One of the most prominent features of the rock is the variable but generally high fossil content. Much of the rock is formed wholly of fossils in a dense and well cemented mass. Brachiopods compose the larger percentage of the fossils.

The shale in the Trenton Formation is generally highly calcareous. The hardness, fissility, and color of the rock varies with the carbonate content. When fresh, the color varies from medium gray to black, and in outcrops weathers to various shades of brown. The shale occurs as irregular thin beds, laminae, and blebs. It is occasionally sheared, and even

in a fresh condition will disintegrate upon slight pressure into thin slickensided flakes. This phenomenon appears to occur in the less calcareous shales.

The calcareous-argillaceous relationship of the two rock units varies considerably, however, in a fresh or weathered condition the rocks are easily seen as either limestone or shale. In fresh cores, the contact between the shale and limestone is quite distinct and tight.

A soft blue clay was found interbedded in shale in the top part of the bedrock in one of the test pits in the borrow area downstream and in one test pit in the foundation area. The clay appears to have limited lateral and vertical extent as it was only encountered at two locations, and core losses that could indicate its presence in core holes occurred only in the upper few feet of rock. The clay possibly owes its origin to the localized leaching of the carbonate from calcareous shale or argillaceous limestone.

Geologic cross sections in the embankment and spillway areas are shown on Exhibit 3.

Foundation Conditions: The cross section of the damsite along the embankment centerline (Section C-C, Exhibit 3) shows a slightly asymmetrical valley. The bedrock surface almost coincides with the ground surface on the left abutment and in the channel, and underlies the ground surface on the right abutment at depths up to 15 feet. There is no reason to expect buried channels or more than minor irregularities in the top of rock profile.

All 10 core holes in the foundation area were terminated in unweathered rock which pressure tests showed to be tight. Stained joints and bedding planes in the holes were found to depths of 3 to 32 feet, with the greater depths being in the center of the valley and on the left abutment. Core losses were limited largely to the upper few feet of weathered rock. The average depth below which no core losses were encountered in the 10 holes is 11 feet below top of rock, excluding a few minor core losses resulting from a stub of core being left in the bottom of the hole on the last run. At shallow depths the core losses are largely caused by the presence of clay filled fractures, severely weathered shale, and to a lesser extent, soft clay interbedded in the upper few feet of rock (DH-11). At greater depth, core losses are largely due to the presence of severely sheared zones in the shale and close jointing occasionally present in the limestone interbeds.

The frequency of occurrences of stained joints and bedding planes, and the water losses during pressure tests, decreased rapidly with depth so that fresh tight rock is found at a relatively shallow depth. The average depth below which no water losses were recorded is 15 feet below top of rock. The range in depth was from one foot in the right channel

area (DH-10) to 33 feet in mid-channel (DH-1). The latter core hole had 100% core recovery and no pressure test losses except as noted in one open fracture at a depth of 33 feet and packer leakage at 25 feet. Since packer leakage occurred fairly frequently, it is probable that the permeable zone in the bedrock is several feet less than the 15 foot average mentioned above.

A cross section along the alignment of the service spillway tunnel (section D-D, exhibit 3) passes through drill holes 1, 2 and 3. The top of rock was encountered at an average depth of about two feet in the holes, and firm rock was encountered at an average depth of about 4 feet.

A cross section along the alignment of the emergency spillway (section E-E, exhibit 3) passes through the general vicinity of core holes 9, 12 and 14, and several test pits. The top of rock along the proposed alignment was found at an average depth of about 5 feet and firm rock was found at a depth of about 15 feet in core holes 9 and 12.

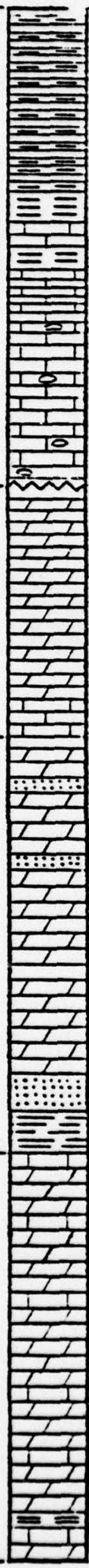
Groundwater Conditions: Groundwater levels measured during the field investigation phase appear to be reliable for the 10 holes located within the proposed embankment area, and somewhat less reliable for the four holes located in the emergency spillway area. The data collected show a close coincidence of the groundwater elevation and the top of rock elevation in the valley bottom and the lower two-thirds of the right abutment. The groundwater level is at a general elevation of about 2400 feet in the valley bottom and rises to a maximum of 2418 feet on the left abutment, and a maximum of 2443 feet on the more gentle right abutment.

Groundwater elevations, as measured in the spillway area above the right abutment, were erratic due to the rapidity with which some of the holes were drilled and the lack of time available to await water equilibrium conditions in the holes. It is probable that the measured water levels were below the natural level in the three holes drilled along the spillway alignment.

GEOLOGY OF THE RESERVOIR

Topography and Overburden: At maximum pool elevation of 2452 feet the reservoir will extend upstream from the damsite about 3600 feet. At the conservation level of 2436 feet, the reservoir length will be about 2700 feet. The pool has a rather elongate regular shape with a maximum width of about 600 feet near the downstream end of the pool. The reservoir has one notable inlet from the east in the upper half of the reservoir. The proposed reservoir and Lincolnshire Branch trend in a north-south direction, crossing the trend of the rock strata in the area at an angle of about 65 degrees.

Table 1



Age	Depth	Formation
Middle Ordovician		Trenton Formation - Thin bedded argillaceous limestone and calcareous shale
	500	Eggleston Formation - Mudstone and limestone
	580	Moccasin Formation - Red calcareous mudstone and argillaceous limestone
	830	Witten Formation - Limestone, fossiliferous, thin bedded
	910	Gratton Formation - Limestone, platy
	990	Cliffield Group of Formations - Limestone - sometimes cherty
Lower Ordovician	1515	Beekmantown Formation - Dolomite, with some limestone
Upper Cambrian	2325	Copper Ridge Formation - Dolomite, with some interbedded sandstone
	3515	Nolichucky Formation - Shale
Middle Cambrian	3650	Honaker Formation - Dolomite with some interbedded limestone and minor shale.
	4950	

UPPER CLINCH WATERSHED
SITE NO. 8

GENERALIZED COLUMNAR SECTION

HARZA ENGINEERING CO., CHICAGO
APPROVED____________

DATE DWG.NO.

Slopes bordering the reservoir are gentle, reaching a local maximum of about 20 degrees in the middle of the reservoir. The overburden in the bottom of the valley underlying the reservoir is thin, averaging perhaps 2 feet downstream and less in the upstream area. The overburden in the slopes bordering the reservoir reaches a maximum thickness of 15 to 20 feet in the proposed impervious borrow areas in the downstream half of the reservoir. In the remainder of the reservoir the overburden averages less than two feet in thickness, and rock outcrops occur over much of the slopes.

Vegetation within the proposed reservoir consists of a few trees and some bushes in the valley bottom and lower slopes in the upstream half of the reservoir. The only vegetation in the downstream half of the reservoir is grass. The entire reservoir area is utilized as pastureland.

Bedrock Stratigraphy, Structure and Lithology: The rocks underlying the reservoir are of Ordovician age, and consist of limestone and calcareous shale and mudstone. Because of the susceptibility of calcareous rocks to solutioning by water and the development of cavernous conditions the geology of the reservoir and adjacent areas was mapped in some detail, with particular attention given to depressions, springs and other surface phenomenon that could be indicative of cavernous conditions.

The formations that underlie the damsite and reservoir are shown on Table 1. Section B-B, exhibit 3, is a geologic section along the long axis of the reservoir.

The damsite and the downstream third of the reservoir is underlain by the Trenton Formation of thin interbedded argillaceous limestone and calcareous shales. The lithology and stratigraphy of the formation have been described in detail in a preceding section of this report. The thickness of the formation in the Burkes Garden Quadrangle does not appear to have been determined due to complex folding, faulting and a general lack of exposures. It is probable that about 500 feet of the formation underlies the damsite. The thickness may increase appreciably for some distance south of the damsite where the formation outcrops in several road cuts.

A close reconnaissance of the Trenton terrane in the reservoir and adjoining areas revealed only very minor solution phenomena. A few seeps in the formation appear to be of superficial origin.

The Eggleston Formation of calcareous mudstone, argillaceous limestone, and some shale, underlies the Trenton Formation. In weathered outcrops, the formation is characterized by a sometimes punky appearance and cuneiform jointing. The cuneiform (wedge shaped) jointing is caused by a three directional linear joint pattern. Another distinctive feature of the formation is interbeds of metabentonite present both in the Eggleston and the bottom of the Trenton Formation. The Eggleston Formation does not

normally form prominent outcrops and is best seen in road cuts. A fair outcrop occurs on the east side of the reservoir adjacent to the creek. The formation strikes N 65°E and dips about 40 degrees to the southeast. The Eggleston is thick bedded and relatively undeformed as compared to the overlying Trenton which is thin bedded, closely folded, and distorted. In a road cut along highway 61, 1/4 mile north of Benbolt, the Eggleston Formation can be seen in contact with the Trenton Formation. The upper 20 feet of the Eggleston in the road cut is deformed and clearly interfolded with beds of the Trenton, although this does not appear true of the contact in the reservoir area.

Two core holes were drilled in the reservoir area through the Trenton-Eggleston contact in order to explore the physical condition, primarily the permeability, of the suspected disturbed contact. The first hole, DH-15, was drilled at an angle of 45 degrees, approximately perpendicular to the bedding; a second hole, DH-16, was drilled vertically when caving in the first hole prevented complete pressure testing. The Trenton-Eggleston contact is located at vertical depths of about 14 and 21 feet in the two holes. The Trenton rock recovered in the cores is similar in lithology and structure to that recovered in the damsite area, i.e., thin bedded limestone and shale, with a variable but generally near vertical dip. The underlying Eggleston beds consist of the rock units described earlier, in strata with a uniform dip, and apparently undisturbed. The pressure test in DH-16 resulted in only moderate water loss to a depth of 20 feet, and no losses below that depth. Thus, in the project area, the Eggleston-Trenton contact does not appear to be a disturbed contact, nor does the contact appear to have more than a minor permeability at shallow depths.

The Moccasin Formation of red, sandy, calcareous mudstone and argillaceous limestone underlies the Eggleston. The dip and thickness of the Moccasin is somewhat variable. Where the formation crosses the proposed reservoir, the stratigraphic thickness is about 75 feet, and due to localized warping the dip varies from 20 to 50 degrees. The overall formational strike is about N 65°E and the dip about 35 degrees to the southeast in the area shown on Exhibit 1. The formation is characterized largely by its bright red colors and prevalence of fossil mud cracks and ripple marks on exposed bedding planes. The formation forms prominent outcrops and is easily mapped in the absence of outcrops by the presence of a red soil cover. No solution phenomena were located in the Moccasin terrane that was reconnoitered in some detail in the area shown on Exhibit 1, or in other surrounding areas that were reconnoitered in less detail.

The upstream third of the reservoir is underlain by a series of limestone formations that belong, in descending order of age, to the Witten and Gratton Formations and the Cliffield Group of formations, all of Middle Ordovition age. The formations have similar lithology and are differentiated largely on paleontological bases. The formations have not been differentiated on Exhibit 1. The limestones in these formations are

relatively pure, but are sometimes cherty or dolomitic. They are generally cryptocrystalline to fine grained, generally medium to thick bedded, and sometimes slabby. The terrain underlain by these formations is characterized by numerous outcrops, rocky slopes, and Karst topography. The Karst topography is particularly prevalent east and west of the reservoir.

The remainder of the Lincolnshire watershed upstream of the reservoir is underlain by the Beekmantown Formation (Lower Ordovician) of dolomite and limestone, Copper Ridge Formation (Upper Cabrian) of dolomite and sandstone, the Nolichuky Formation (Upper Cambrian) of shale and the Honaker Formation (Middle Cambrian) of dolomite. Quartz sandstone forms a very prominent outcrop near the top of the Copper Ridge Formation a few hundred feet upstream of the proposed reservoir. A major spring issues from the Honaker Formation at elevation 2580 feet in the bottom of the valley about one mile upstream of the reservoir.

Watertightness of Reservoir: There appears to be no potential for reservoir leakage through the Trenton, Eggleston and Moccasin formations that underlie the downstream two-thirds of the reservoir. The generally high clay mineral contents of the calcareous rocks in the formations, plus the prevalence of interbedded shales and mudstones appear to prelude the development of solution effects more pronounced than small vugs and other minor effects along joints at shallow depths.

The remainder of the reservoir is underlain by limestones of the Witten, Gratton, and Cliffield Group of formations. These limestones have a Karst topography that is well developed in the higher elevations in the area and much less developed in the lower elevations adjacent to the valley bottoms, indicating that streams such as the Clinch River and the Lincolnshire Branch have valley bottoms below the base level of Karst development in the adjacent hills. The inferred relationship between the Karstic terrane and groundwater flow is illustrated in section A-A, Exhibit 3.

A careful search of the reservoir area revealed an open sink hole in the Witten Formation in the bottom of a small tributary valley on the east side of the reservoir and, as noted above, a spring issuing from an opening in the limestone at the north end of the reservoir. Both of these phenomena are above the water supply pool elevation 2436.

There is no evidence of Karstic development within the water supply pool elevation of the reservoir. As explained earlier, the valley bottom appears to be at or below the level of the Karst development, thus little if any leakage of water from the water supply pool to the adjacent groundwater basin to the west is expected. However, if significant leakage occurs through open sink holes that are not now apparent, remedial measures to seal the areas of leakage should not be difficult or expensive because of the small size of that part of the reservoir within the Karstic limestone.

SOURCES AND CHARACTERISTICS OF CONSTRUCTION MATERIALS

All of the natural earth materials required for construction of the embankment are available within a short distance of the damsite. Approximately 113,000 cubic yards of impervious material is available from overburden in borrow areas "A" and "B" on the lower slopes of the reservoir within 1200 feet of the embankment. Assuming a rippability depth of 3 to 6 feet, 45,000 to 90,000 cubic yards of weathered rock material suitable for random fill is available under the overburden. Approximately 19,000 cubic yards of random fill material will be obtained from the emergency spillway excavation. Approximately 75,000 cubic yards of impervious material is also available from borrow area "C", on the right slope downstream and within 1,000 feet of the embankment.

Concrete aggregate and riprap of excellent quality can be obtained from the Pounding Mill Quarry at Pounding Mill, Virginia, 12 miles west of the damsite. Crushed limestone from the same source can be used for filter material. The quarry is operated primarily for lime processing and crushed rock is sometimes a by-product.

SOILS INVESTIGATIONS

As described earlier in this report, undisturbed and disturbed samples of overburden materials were taken for laboratory testing. Testing included determinations of moisture, natural density, specific gravity, Atterberg Limits, permeability, unconfined compression strength, gradation, consolidation characteristics, triaxial shear strength, and compaction characteristics. A summary of the test results is shown on sheet 4 of Exhibit 9. Test procedures and the detail test results are also presented in Exhibit 9.

FOUNDATION CHARACTERISTICS

The overburden foundation characteristics were investigated by testing block samples from test pits 8, 21, 22, 24 and 26 and by testing shelby tube samples from drill holes DH-10 and 11. Standard penetration tests were performed in all drill holes at the dam site.

The characteristics of the silty clay overburden at the dam site are summarized in the following table:

Overburden Foundation Characteristics

	Max.	Min.	Avg.
1. Classification tests			
a. natural moisture, %	31.0	8.8	23.0
b. natural dry density, pcf.	115.0	90.6	97.8
c. Atterberg limits			
Liquid Limit, %	47.2	33.7	39.2
Plasticity Index, %	20.2	12.9	16.1
d. specific gravity	2.72	2.69	2.70
e. gradation, % passing #200 sieve	93	50	80
2. Permeability, 1 test	3.16×10^{-8} cm/sec		
3. Strength			
a. unconfined compression, tsf.	5.52	1.87	3.58
b. triaxial shear (consolidated-undrained), 1 test	$\phi' = 25.0°$ $C' = 0.4$ tsf.		
c. standard penetration test blows/foot	31	8	20
4. Consolidation, 1 test	0.03 ft/ft (loading from 0.3 to 2.8 tsf.)		

Based on the above data and on visual observation, the overburden at the dam site is a brown to reddish-brown silty clay (CL to CL-ML), stiff to very stiff, with low permeability and compressibility.

The overburden at depth grades into highly weathered rock and rock fragments which in turn grades into fresh rock. The foundation, including the overburden will be able to support the load of the dam.

BORROW MATERIAL CHARACTERISTICS

Bag samples of overburden materials were taken at the sites of the proposed borrow areas. The materials sampled have similar characteristics as the overburden at the dam site. A summary of the test results is presented in the following table. Since the differences between borrow areas is slight, the test data obtained at all proposed borrow areas is summarized together.

Borrow Material Characteristics

	Max.	Min.	Avg.
1. Classification tests			
a. natural moisture, %	29.1	10.1	21.4
b. Atterberg Limits			
Liquid Limit, %	62.0	33.6	43.5
Plasticity Index, %	35.3	10.7	20.8
c. specific gravity, 2 tests	2.69 and 2.71		
d. gradation, % passing #200 sieve	96	77	89
2. Permeability at 95% max. dry density, 2 tests	1.7 and 4.7 x 10^{-7} cm/sec		
3. Triaxial shear strength (consolidated-undrained) at approx. 95% max. dry density, 2 tests	Ø' = 24.5° and 22.0° c' = 0.4 and 0.5 tsf.		
4. Consolidation at 95% max. dry density, 1 test	0.02 ft/ft (to loading of 2.0 tsf.)		
5. Compaction			
optimum moisture, %	25.0	19.0	22.4
max. dry density, pcf.	103.0	90.5	97.6

Natural moisture contents are close to the optimum moisture content. Moisture control during construction of the embankment will not be difficult.

PRELIMINARY DESIGN RECOMMENDATIONS

Soil Design Parameters: The following parameters for the overburden foundation and borrow materials are suggested for design. These values are based on the field and laboratory data as summarized above:

Design Parameters

	Foundation	Embankment
1. Unit weight		
moist	120 pcf.	120 pcf.
saturated	125 pcf.	125 pcf.
2. Permeability	Cutoff through overburden	$3x10^{-7}$cm/sec
3. Shear strength (effective stress parameters)	$\phi' = 25°$ $c' = 0.4$ tsf.	$\phi' = 24°$ $c' = 0.4$ tsf.
4. Compressibility	0.03 ft/ft	0.02 ft/ft

Foundation Treatment and Drainage: The organic soil will be stripped from the foundation area of the dam. This stripping is not expected to average more than one foot in thickness. It will be slightly thicker in the valley bottom than on the abutments.

Seepage control in the foundation will consist of a core trench to the top of unweathered and relatively tight rock. Based upon pressure test data in core holes in the embankment area, relatively tight rock was found at an average depth of about 20 feet below the ground surface. The depths of the core trench will vary from about 5 feet to 25 feet, of which 2 to 15 feet will be overburden. The approximate depths of the core trench at the several drill holes along the center line of the dam are summarized below:

Hole #	Elev. Top of Hole	Approximate depth of core trench, ft.
DH-4	2462	0' (hole is above top of dam)
DH-8	2401	5'
DH-1	2402	15'
DH-5	2414	23'
DH-6	2433	22'
DH-7	2459	0' (hole is above top of dam)

Grouting of bedrock beneath the core trench is not anticipated. However, final judgement should be delayed until the core trench is excavated and the base of the trench is inspected.

The principal spillway will be founded on competent rock which will be found at an average depth of about 4 feet. The emergency spillway will be excavated in rock for much of its length.

Embankment design will include a trench drain located downstream of the core trench and parallel to it. The drain will extend through the overburden and into the weathered rock.

Embankment Stability Analysis: A preliminary evaluation of the stability of the embankment and its foundation was made, using the above design parameters and assuming an essentially homogeneous embankment with a 3h:lv. slope upstream and a 2.5 h:lv. slope downstream. The method of analysis outlined by A. W. Bishop and N. Morgenstern in the following paper was used:

"Stability Coefficients for Earth Slopes", Geotechnique, Vol. 10, No. 4, pp. 129-150.

The following factors of safety were computed:

Condition	Factor of Safety
End of construction, 2.5h:lv. slope	2.1
Steady-state seepage (downstream slope)	2.3
Rapid drawdown (upstream slope)	1.8

Based on this preliminary evaluation, an embankment with a 3 h:lv. slope upstream and a 2.5 h:lv. slope downstream will be stable under all loading conditions.

FINDINGS AND CONCLUSIONS

Field investigations and the results of laboratory soils tests indicate that the overburden and bedrock in the dam site area will suffice, with treatment, as a foundation for hte proposed embankment. The treatment will consist of the removal of the thin organic soil cover and the construction of a core trench. A trench drain will be placed downstream and parallel to the core trench. Grouting below the core trench is not anticipated at this time.

The principal spillway will be founded on weathered but competent rock. Portions of the emergency spillway will be excavated through relatively fresh rock.

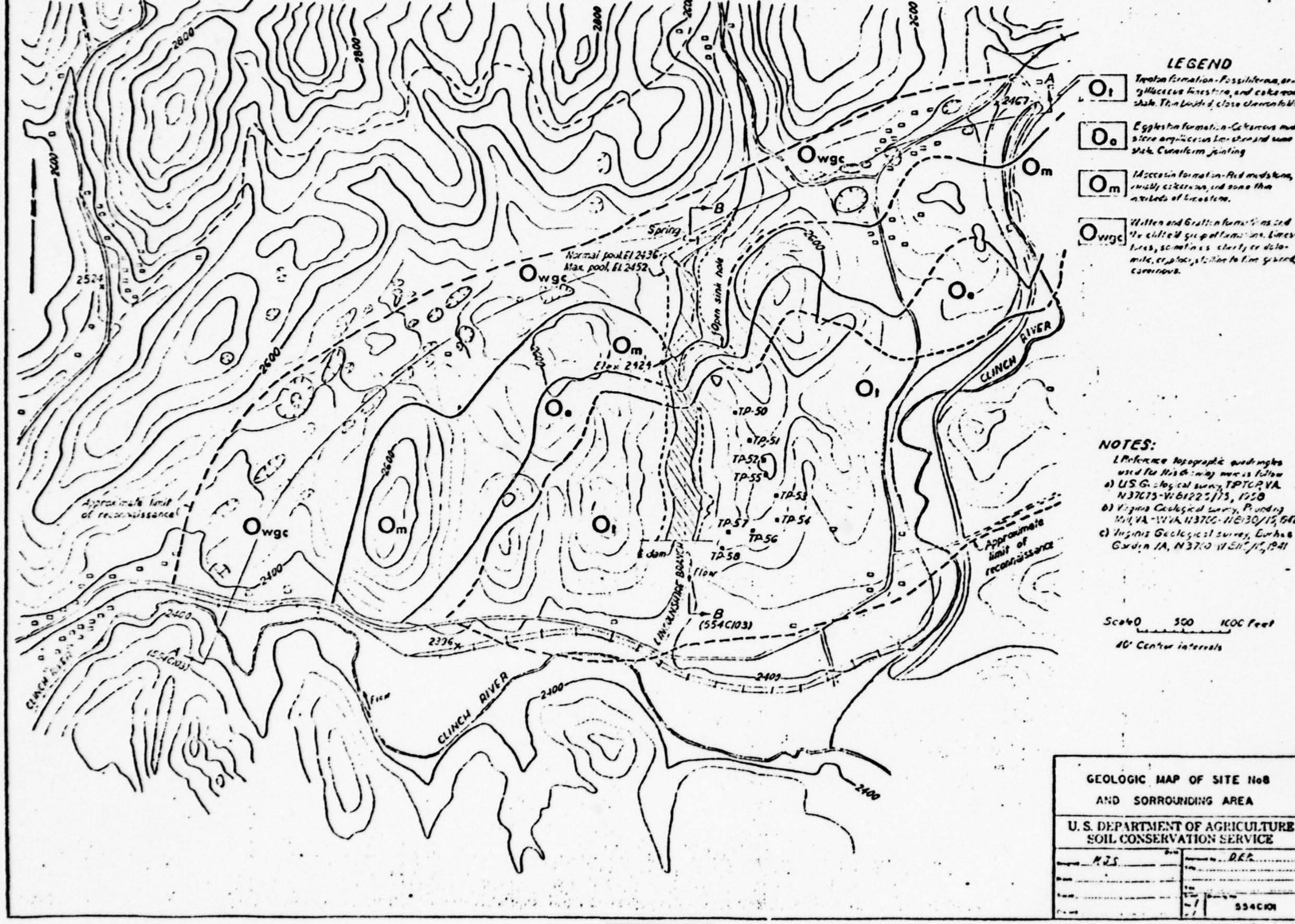
Sufficient quantities of impervious and random borrow materials for the embankment appear to be available in the spillway and core trench excavations and in the proposed reservoir area within 1200 feet of the embankment. Additional materials were explored in areas downstream of the embankment. Concrete aggregate, rip rap, and filter material are available from commercial sources.

Preliminary stability computations of the dam show that a 3h:1v. upstream slope and a 2.5 h:1v. downstream slope are stable.

There appears to be no potential for reservoir leakage through the Trenton, Eggleston and Moccasin formations that underlie the downstream two thirds of the reservoir below an elevation of about 2424 feet. No solution effects were observed in these formations except for small vugs and slight solution effects along some surficial joints.

The limestones which underlie the upstream one third of the reservoir have a well developed Karst topography outside of the reservoir area and a minor Karst development within the reservoir above the water supply pool level. The results of our studies indicate that there will be no leakage from the water supply pool to adjacent valleys. If however, some leakage should take place when the reservoir is full, it is believed that local remedial treatment can be effected.

Exhibit 1


LEGEND
Eggleston formation
Moccasin formation
NOTES:
Scale 0 500 1000 Feet
Normal pool El 2436
Max. pool El 2452
Spring
Open sink hole
Elev 2429
TP-50
TP-51
TP-52
TP-53
TP-54
TP-55
TP-56
TP-57
TP-58
flow
B
(554C103)
CLINCH RIVER
Approximate limit of reconnaissance
2400
2600
2800
2467
2524
2336
GEOLOGIC MAP OF SITE No8
AND SORROUNDING AREA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
554C101

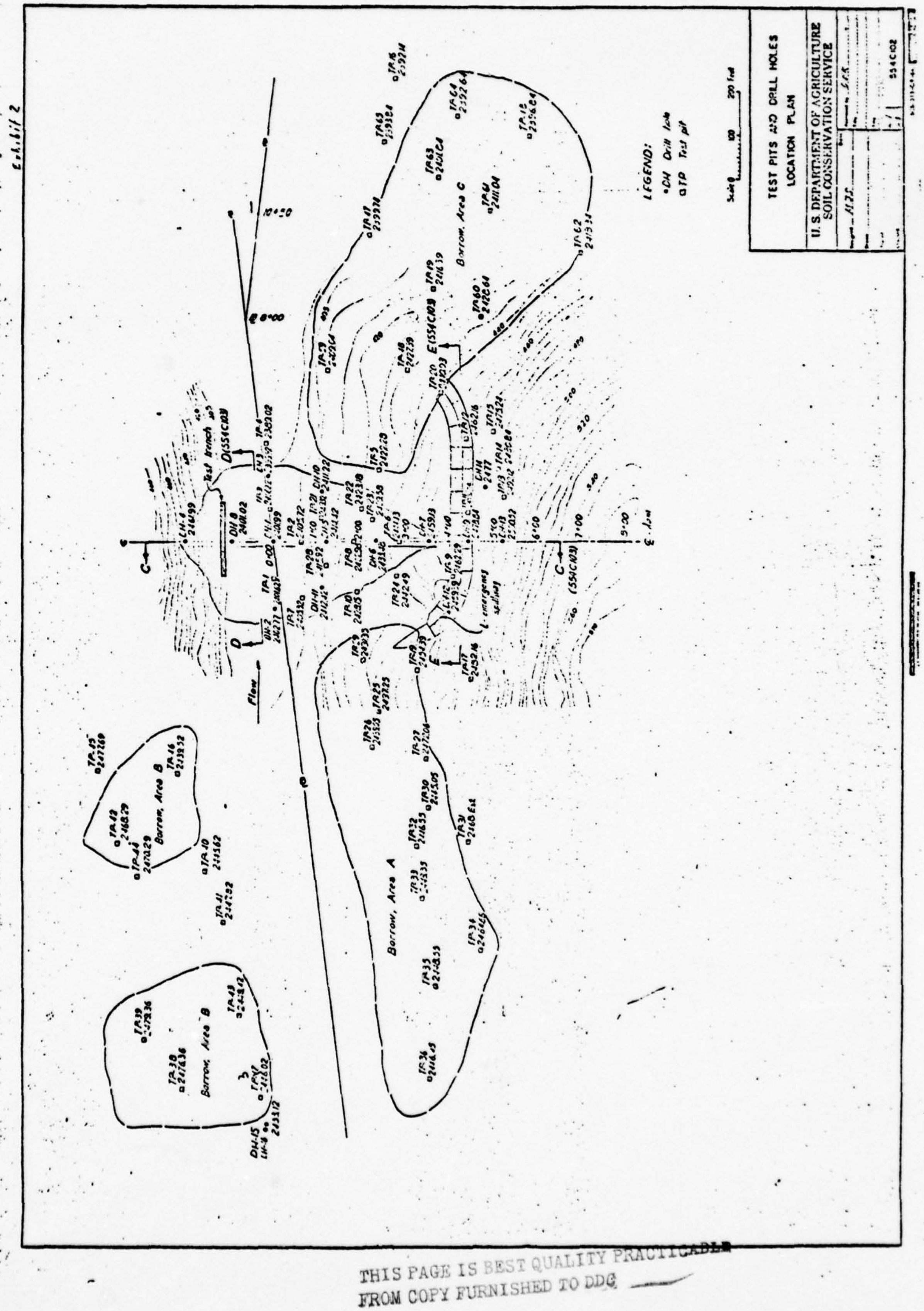
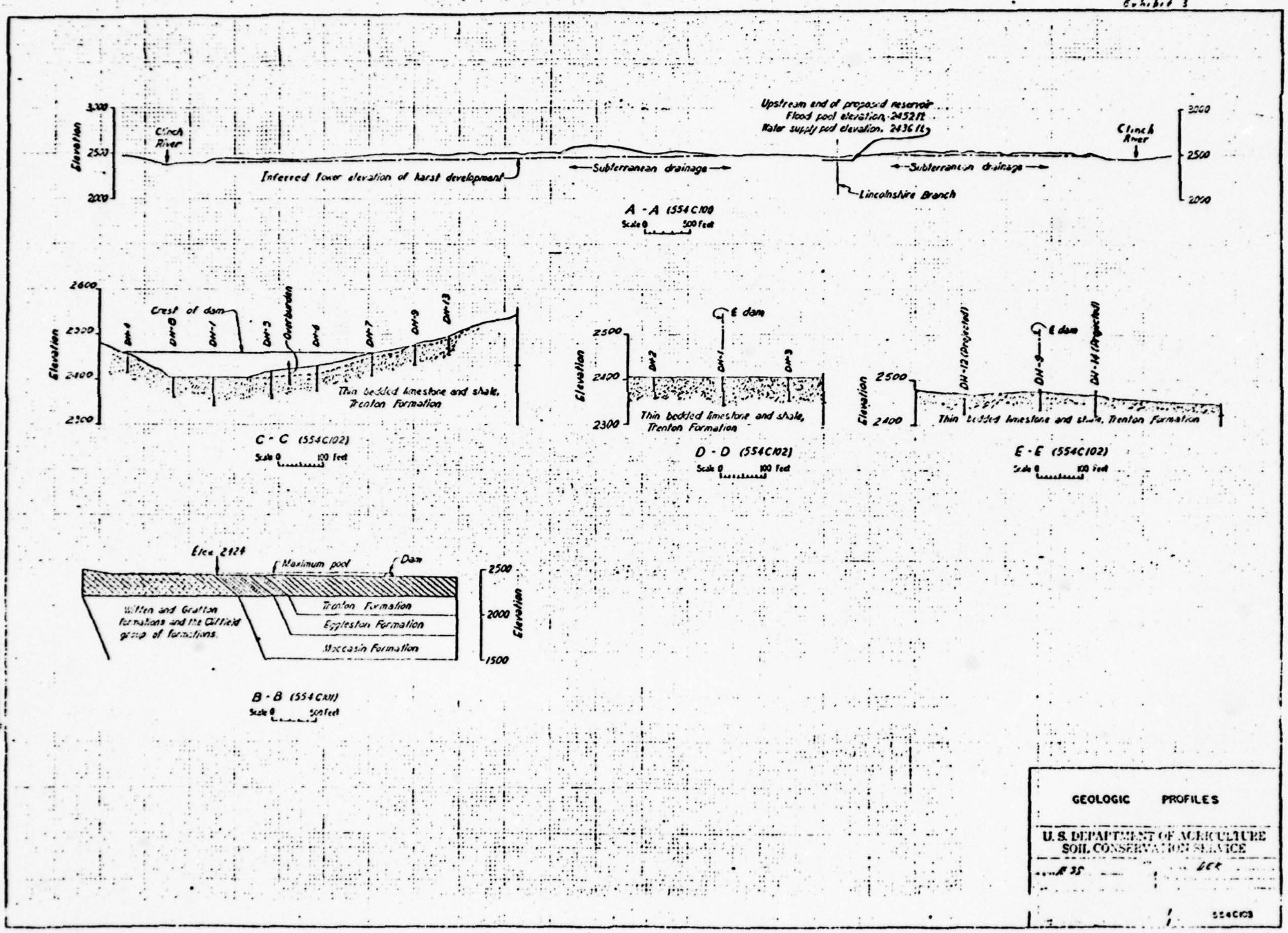
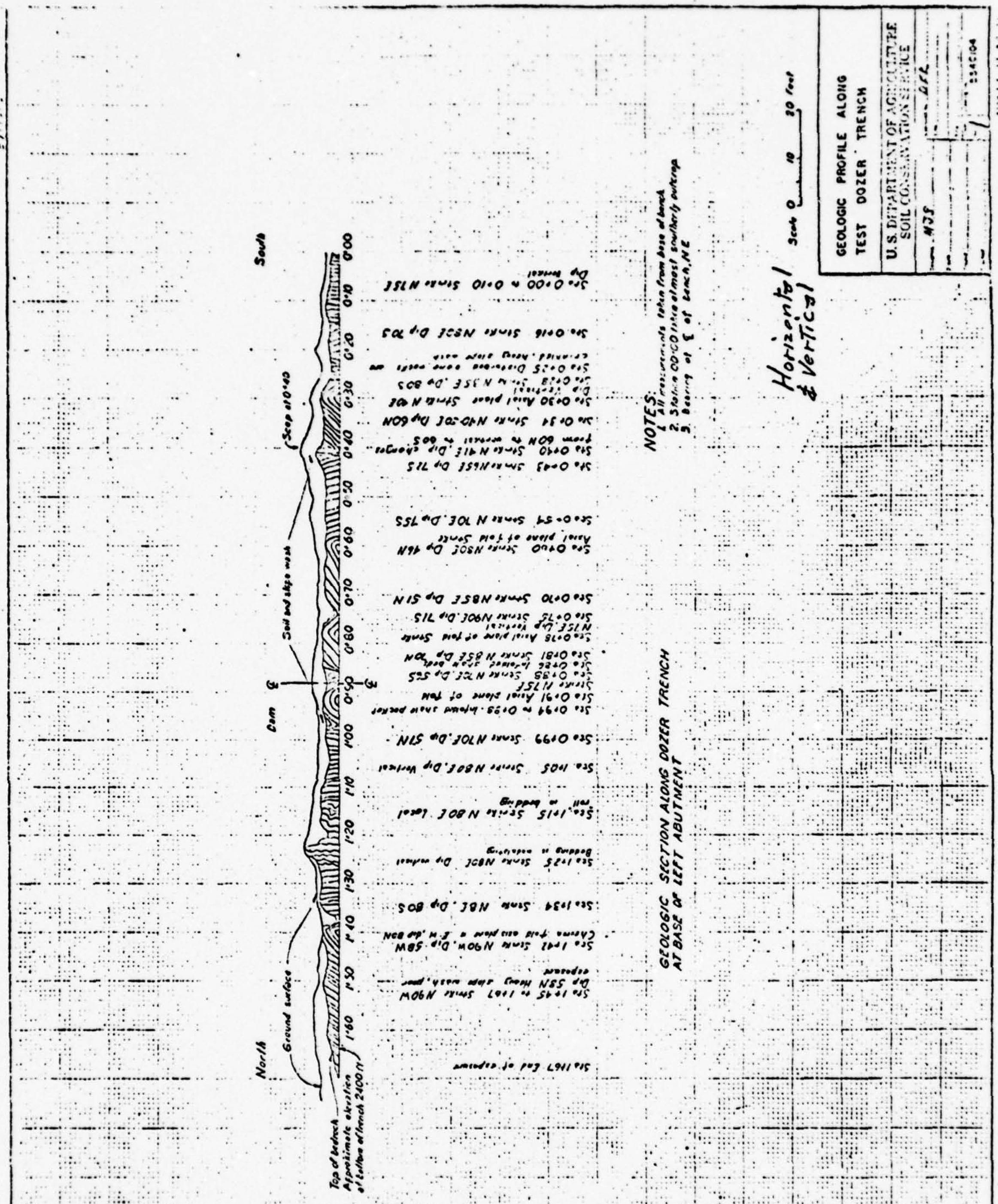


Exhibit 2
TEST PITS AND DRILL HOLES
LOCATION PLAN
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
LEGEND:
DH Drill hole
TP Test pit
Borrow, Area A
Borrow, Area B
Borrow, Area C
Test trench
Flow

Exhibit 3


Elevation
Clinch River
Upstream end of proposed reservoir
Flood pool elevation, 2452 ft
Water supply pool elevation, 2436 ft
Inferred lower elevation of karst development
Subterranean drainage
Lincolnshire Branch
A - A (554C100)
Scale 0 500 feet
Crest of dam
Overburden
DH-4
DH-0
DH-1
DH-3
DH-6
DH-7
DH-9
DH-13
Thin bedded limestone and shale, Trenton Formation
C - C (554C102)
Scale 0 100 feet
℄ dam
DH-2
D - D (554C102)
DH-12 (Projected)
DH-14 (Projected)
E - E (554C102)
Elev. 2424
Maximum pool
Dam
Witten and Gratton formations and the Clifield group of formations
Trenton Formation
Eggleston Formation
Moccasin Formation
B - B (554C101)
GEOLOGIC PROFILES
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
554C103


GEOLOGIC PROFILE ALONG TEST DOZER TRENCH
U.S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE
GEOLOGIC SECTION ALONG DOZER TRENCH AT BASE OF LEFT ABUTMENT
NOTES:
Horizontal & Vertical
South
North
Dam
Ground surface
Soil and slope wash
Top of bedrock

APPENDIX V

STABILITY CALCULATIONS

FOR USE ON U.S. GOVERNMENT WORK ONLY

HARZA ENGINEERING COMPANY CHICAGO	SUBJECT DAM Stability Analyses COMPUTED BL CHECKED	PROJECT Upper Clinch Dam # FILE NO. 544 C DATE 12/1/70 PAGE 1 OF 1 PAGES

Purpose : Analyse the slope stability of the dam.

References : 1. SCS Work drawings and maps

2. HECO Upper Clinch Valley Watershed Dam #9, Geologic and Soils Investigations, Oct. 1970

3. DEK Triaxial shear strength, etc 10/7/70 Computation

4. HECO HARZA/MORGEN. A Computer Program for Stability Analysis of Slope Using the Morgenstern - Price Method.

Assumptions :

1. Max. height of 57.7 foot dam was analysed.

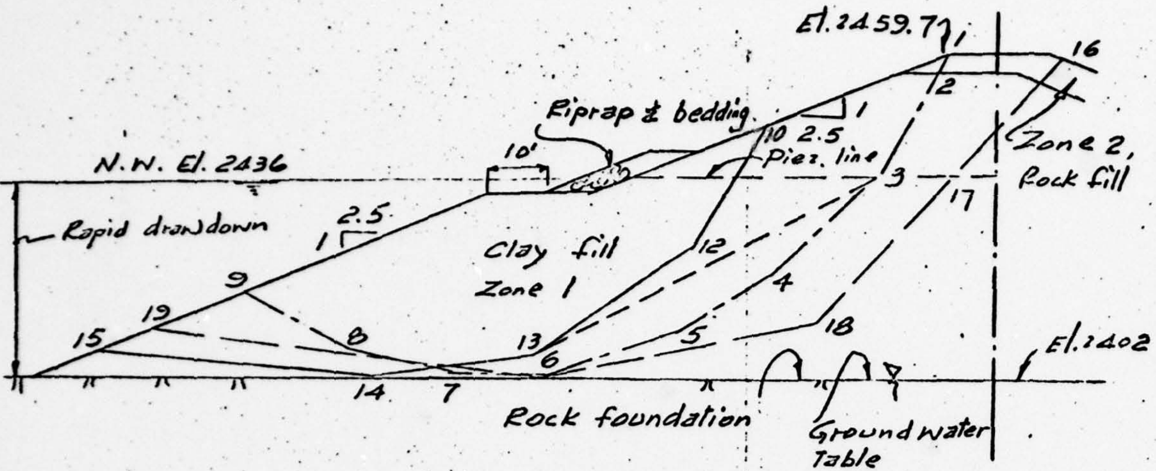
2. the foundation is rock, ground water table is at top of rock.

3 other assumptions as stated on attach SKC 401

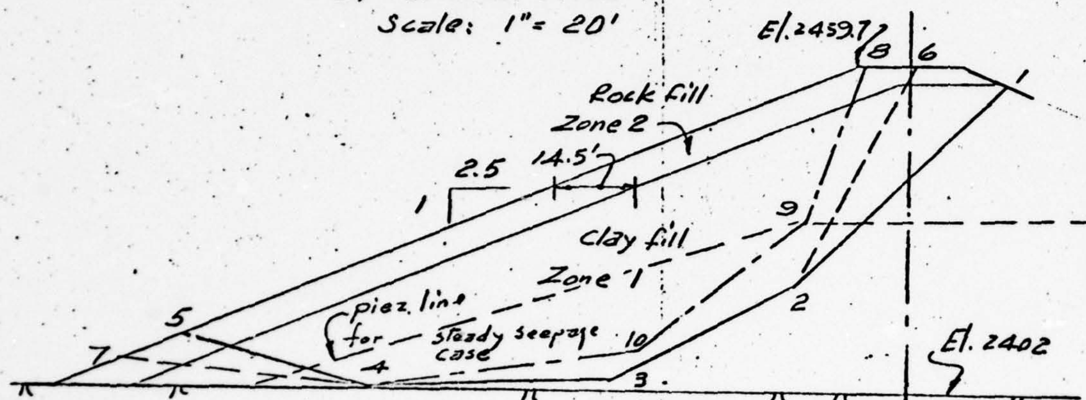
Results :

Factors of safety for different cases are summarized on SKC 401. These are based on 2.5:1 slopes both on upstream and downstream.
Per SCS, they preferred the upstream face has 3:1 slope from El. 2456 down. We know this change will increase the f.s. of safety.

THIS PAGE IS BEST QUALITY PRACTICABLE
FROM COPY FURNISHED TO DDC



UP-STREAM FACE
Scale: 1" = 20'



DOWN-STREAM FACE
Scale 1" = 20'

Note: Computer program based on Morgenstern and Price was used for computation, the output sheets are kept in HECO file

FACTORS OF SAFETY

Failure surface	Up-stream face, rapid drawdown	
	W/O Earthquake	W/ Earthquake
1-2-3-4-5-6-7-8-9	2.64	1.98
10-12-13-14-15	2.61	1.90
16-17-18-6-19	2.42	1.77
1-2-3-13-14-19	2.39	1.78

SOIL PARAMETERS

Soils	γ_m pcf	γ_{sat} pcf	ϕ'	c' psf	r_u	α	
Zone 1	120	125	24°	200	*0.4	0.1	
Zone 2	128	130	33°	0	0.0	0.1	

* For during construction only

FACTORS OF SAFETY

Failure surface	Down-stream face		
	During Construction	Steady seepage W/O Earthquake	Steady seepage W/ Earthquake
1-2-3-4-5	1.93	2.35	1.81
6-2-3-4-7	1.92	2.40	1.85
8-9-10-4-5	2.05	2.57	2.00

Legend:
γ_m: moist unit weight
γ_{sat}: saturated unit weight
r_u: Pore pressure ratio
α: Earthquake factor

B.L.

UPPER CLINCH DAM #2
SUMMARY OF
STABILITY ANALYSES
HARZA ENGINEERING CO., CHICAGO
APPROVED ....
DATE 12/14/72

K&E CO. 19-1233

APPENDIX VI

REFERENCES

APPENDIX VI

REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, (Washington, D.C., Department of the Army, Office of the Chief of Engineers).

2. HEC-1, Flood Hydrograph Package, Hydrologic Engineering Center, U.S. Army Corps of Engineers, January 1973.

3. Design of Small Dams, U.S. Department of the Interior, Bureau of Reclamation, Second Edition, 1973.

4. "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian," Hydrometeorological Report No. 33, U.S. Weather Bureau, April 1956.

5. "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, U.S. Weather Bureau, May 1961.

6. Upper Clinch Valley Watershed Project, Multiple Purpose Dam No. 8, Harza Engineering Company Design Report, May 1971.

APPENDIX VII

HYDRAULIC/HYDROLOGIC DESIGN INFORMATION

COMPUTATION SHEET
SCS-522 REV 5-58

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE VIRGINIA		PROJECT UPPER CLINCH VALLEY DAM 42	
BY RAH	DATE 5/70	CHECKED BY	DATE	JOB NO. VA-598-H
SUBJECT DESIGN INFORMATION			SHEET 1 OF

(1.) Class "C" Water Supply Structure

(2.) a. Two stage Riser
b. The Riser was planned as a single stage baffled 30" pipe. To avoid using a baffled pipe, the crest of the riser was set D/2 below the emergency spillway crest, with an orifice opening at the water supply pool elevation which will discharge the required capacity that was originally planned using the baffled pipe.

(3.) Sediment Storage

a. 155 Ac. Ft. in sediment pool
b. 10 Ac. Ft. aerated

(4.) Set crest of riser D/2 below the emergency spillway crest.

(5.) Set crest of emergency spillway at work plan elevation.

a. Check min. required storage by TR 10.
b. Use TR 40, MC II, Cn 70, 1% chance of use.

(6.) Use a 30" I.D. Reinforced concrete pipe.

(7.) 7.5' x 2.5' riser I. D. reinforced concrete

(8.) Emergency spillway bottom width ~~50~~ 55 ft. side slope 2:1, inlet slope 2%, outlet slope 2%, 30 feet level section.

(9.) The embankment slopes 2.5:1 over 3:1 upstream with two berms upstream at the water supply pool and sediment pool. The berm at the water supply pool will be 10 feet with a one ft. drop. The berm at the sediment pool elevation will be 10 ft. and level. The slope downstream will be 2.5:1.

(10.) [illegible] TVA [illegible]
TVA [illegible]
[illegible]

APPENDIX VIII

PREVIOUS INSPECTION REPORTS

December 13/74

ANNUAL OPERATIONAL AND MAINTENANCE INSPECTION FOR SITE # 8, UPPER CLINCH VALLEY WATERSHED PROJECT, TAZEWELL , VA.

Site #8 was inspected 12/13/74 by Larry Goff, District Conservationist, Tom Tibbs, Soil Conservationist, and Clifford Necessary, Tazewell Town Manager. This is the only structure completed in this project.

The structure is operating as planned. There are no obvious problems concerning the operational status.

The structure is being maintained adequately. The dam, spillway and area below the dam has a good vegetative cover. The borrow areas upstream from the dam are healing satisfactorily. It is estimated that by next fall an adequate vegetative cover will be present in these areas.

The only areas that need attention are a few washes at the outer slope of the emergency spillway. It is agreed by the sponsor and SCS that these areas be sodded with grass next spring to stabilize these areas.

Larry Goff

Larry Goff, District Conservationist

Clifford Necessary, Tazewell Town Manager

X

OPERATION AND MAINTENANCE INSPECTION REPORT
FOR LINCOLNSHIRE DAM

Site No. 8 — Upper Clinch Valley Watershed

Type of Inspection: Annual X Special ____ Date April 12, 1976

ITEM	CONDITION Satis-factory	CONDITION Unsatis-factory	Describe Maintenance and Needed Repairs	Esti-mated Costs	Agreed date Repairs to be Completed
Vegetation	X				
Embankment	X				
Principle Spillway	X				
Emergency Spillway		X	Small gulleys at exit need to be stabilized	$50.00	June 30
Impact Basin	X				
Toe Drains	X				
Fences	X				
Impoundment Area	X				
Other					

REMARKS: Small gulleys at spillway exit will be backfilled and seeded.

Concurred in by:

Clifford R. [illegible]
Sponsor Representative

Donald P. Hiser
SCS Representative

Distribution:

No. of copies

1 - Town of Tazewell
3 - Soil Conservation Service
1 - Tazewell County Soil and Water Conservation District
1 - Tazewell County Board of Supervisors

OPERATION AND MAINTENANCE INSPECTION REPORT
FOR LINCOLNSHIRE DAM

Site No. 8 — Upper Clinch Valley Watershed

Type of Inspection: Annual ✓ Special ______ Date 10-14-76

ITEM	CONDITION Satis-factory	Unsatis-factory	Describe Maintenance and Needed Repairs	Esti-mated Costs	Agreed date Repairs to be Completed
Vegetation	✓				
Embankment		✓	Spot seed, fert.		April 30 77
Principle Spillway	✓				
Emergency Spillway		✓	Needs stabilization		April 30 77
Impact Basin	✓				
Toe Drains	✓				
Fences	✓				
Impoundment Area	✓				
Other					

REMARKS: ______________________________

Concurred in by:

______________________________ Sponsor Representative

______________________________ SCS Representative

Distribution:

No. of copies

1 - Town of Tazewell
3 - Soil Conservation Service
1 - Tazewell County Soil and Water Conservation District
1 - Tazewell County Board of Supervisors

SAFETY INSPECTION OF SITE 8
LINCOLNSHIRE BRANCH, UPPER CLINCH VALLEY
WATERSHED, TAZEWELL, VIRGINIA

A field inspection of Site 8, Upper Clinch Valley Watershed was made on April 20, 1978 by Duncan C. McGregor, Area Engineer, SCS, Marion, Virginia.

The following items were examined and their condition noted.

1. Embankment Structure

   a. Settlement - No visible settlement, localized or widespread, anywhere on the embankment.

   b. Slope Stability - No visible indications of any slope movement or impending movement.

   c. Seepage - No evidence of any existing or past seepage on embankment slopes and toes, abutments, embankment and abutment contacts, and downstream valley areas.

   d. Drainage Systems - Both foundation drains were flowing freely with no obstructions at the outlet. The water was clear and flowing at a rate within normal limits for the system.

   e. Slope Protection - No active rills or gullies were visible on the embankment slopes. Vegetative cover, overall, is very good.

      No damage from wave action was evident on the embankment.

      Two well defined paths have developed on the downstream face of the embankment. They have been created by constant foot traffic that originates in the recreation complex below the embankment. One in in the center of the embankment and one is at the contact of the embarkment and left abutment. At the time of the inspection no rills or gullies were developing along these paths. Some traffic control or durable protection is needed on these areas to prevent further damage to vegetation and formation of gullies.

2. Emergency Spillway

   a. Approach and Outlet Channels - The approach and outlet channels are open and free to function as planned during emergency spillway flow. A small storage shed was built in the flood pool in the vicinity of the approach channel. The size of the shed and distance from the control section prevent it from having any effect on the spillway flow.

   b. Spillway Section - The spillway section is well stabilized by bedrock and vegetation. There is no evidence of erosion,

3. Outlet Works

a. Intake Structure - Concrete surfaces showed no cracking, spalling or deterioration. There was no indication of any structural cracking or movement. The joints were sound with no indication of distress or leakage.

Ungated openings, ladders and trash racks were free of debris and in good operating condition.

These observations were made on portions of structure above the normal pool level.

b. Water Supply Gates and Draw Down Facilities - As far as could be determined from conversation with town employee at the site, the water supply and draw down drain gates have not been tested for operational adequacy since they were installed.

The stem, stem guides and hoist assembly for the two water supply gates appear to be structurally sound and in proper alignment.

The hoist assembly for the draw down drain gate has been displaced several inches off center from vertical. The reason or cause for this is not readily apparent.

The water supply and draw down gates should be tested immediately and any necessary repairs or adjustments made.

c. Conduit - There is no deterioration of the conduit surfaces at the outlet. The visible joints are sound and show no signs of cracking, movement or leakage. The cradle and bent appear sound and show no signs of cracking or movement.

d. Stilling Basin - The stilling basin is operating as intended with no indications of riprap movement or changes in basin slope. The downstream channel is functioning properly with no signs of erosion or scouring on the banks or in the channel bed.

APPENDIX IX

CONDITIONS

APPENDIX IX

CONDITIONS

This Report is based on a visual inspection of the dam, a review of available engineering data and a hydrologic analysis performed during a Phase I Investigation as set forth in the U.S. Corps of Engineers' "Recommended Guidelines for Safety Inspection of Dams" and the contract between the U.S. Corps of Engineers and Gilbert Associates, Inc.

The foregoing inspection, review and analysis are by their nature limited in scope. It is possible that conditions exist which are hazardous, or which might in time develop into safety hazards, that are not detectable by this inspection, review and analysis. Accordingly, Gilbert Associates, Inc. cannot and does not warrant or represent that conditions which are hazardous, or which may in time develop into safety hazards, do not exist.